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of the

AMERICAN CONCRETE INSTITUTE

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AMERICAN CONCRETE INSTITUTE

Founded 1905; incorporated 1906 (National Association of Cement Users); charter amended 1913. (American Concrete Institute.)

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JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

News AMERICAN CONCRETE INSTITUTE Letter

MILWAUKEE—FEBRUARY 24, 25, 26, 1931

PRESIDENT Duff Abrams and the Secretary of the Institute met Milwaukee members and friends at luncheon at the Hotel Schroeder, Milwaukee, March 17, and discussed in a preliminary way the 27th annual convention of the Institute which will be held there Tuesday, Wednesday and Thursday, February 24, 25, 26, 1931.

K. H. Talbot was chairman of the Milwaukee luncheon meeting and he and others made it very plain that Wisconsin will do a great deal to see that the 1931 meeting is a huge success. Others who were present: T. J. Baker, R. E. Brevik, D. R. Collins, A. W. Devos, H. L. Foster, Jack Franklin, H. S. Greene, Walter Hirschberg, Ray Newhouse, E. C. Schumann, C. A. Wiepking.

Mr. Collins, who is president National Concrete Products Association and of the Wisconsin Concrete Products Association (perhaps the most active and best organized of large concrete products groups) is serving temporarily as secretary pending the organization of a Milwaukee Convention Committee. He predicts 300 concrete products manufacturers at the Institute's next convention.

MEMBERS AND MEMBER-GETTERS

JOHN G. AHLERS was top man on the Honor Roll. He sponsored 23 new members in the year ended February 1, 1930. He sponsored 14 new members in the year ended February 1, 1929, when he was in second place on the roll. He is perfectly willing that you know how he does it. In the first place he is thoroughly alive to what Institute membership means. He makes mental (perhaps written) note of men who are interested in various phases of work the Institute is studying. In personal contacts, in correspondence he has

members in mind. He has instilled the key to the membership matter in the minds of at least two young women in his office. So they *see prospects* in the names that drift through correspondence. Mr. Ahlers writes a sample letter or two; asks the secretary of the Institute for pamphlets descriptive of Institute work; membership application blanks. The young ladies do the rest, making sure that the secretary of the Institute gets names and addresses of all prospects, to follow systematically as *Ahlers'* prospects. He made requisition for his prize in cash—this year, $30.00. The young ladies get it. Simple. Others could do it.

W. F. Way has been in prize company before also. He didn't need any more fountain pens; was afraid we might buy him a mustache cup, so, with 13 new members to his credit he took "second money"—$20.00.

I. E. Burks doesn't have his center of activity in a thickly populated region but he is responsible for 10 new members. H. F. Gonnerman, habitually a member-getter, tied with Burks—each won a third prize of $15.00. Last year Mr. Gonnerman was in first place.

W. D. M. Allan came up suddenly. January 22 he had but 3 new members to his credit but on February 1 his total stood 9, just enough to win a cash prize of $10.00.

This year be wary of Wisconsin. Those Milwaukee members, having won the convention for 1931, are out after all sorts of new records—members among them.

The Secretary's office is getting out a new pamphlet—to slip readily into a No. 10 envelope and point out greater membership advantages by reason of the new JOURNAL. Any member may have as many pamphlets as he thinks he can get into the hands of prospective members. The personal touch counts—a man becomes a member because a friend or acquaintance in suggesting membership advantages is more convincing than any literature from the Secretary's office could possibly be. Institute membership should be doubled. How many pamphlets can you use?

BOARD MEETS MAY 10, AT VIRGINIA BEACH

THE Board of Direction will hold its spring meeting at the Cavalier Hotel, Virginia Beach, Virginia, May 10.

Previous to the Board meeting will be meetings of Advisory and Program committees to prepare reports for the Board on plans for the Institute's year; with respect to committee assignments and personnel and the high points and the outline of the 27th annual convention next February 24-26 in Milwaukee.

The Advisory Committee has half a hundred prospective committee tasks—the Advisory committee's immediate task being to find half a hundred ambitious chairmen.

IF YOU have a member prospect in mind ask the secretary to send him a copy of a new pamphlet setting forth new advantages in Institute membership by reason of the JOURNAL. Or ask for a supply of these pamphlets—any number you can use to advantage.

REINFORCED CONCRETE COLUMN TEST FUND

AT THE convention in New Orleans, W. A. Slater, Chairman of Committee 105, Reinforced Concrete Column Investigation, presented a progress report of some of the work being done at the University of Illinois and at Lehigh University, preliminary to the actual testing of nearly 600 columns and explaining somewhat more of the underlying purpose of the tests as influencing future reinforced concrete column design.*

John G. Ahlers, chairman of the New York Committee, under the general chairmanship of Henry C. Turner, in raising the column test fund said:

"It has been an inspiration to get something to work for in a big way. The Institute has an opportunity to do something really worth while in making these tests on columns, and with that as an inspiration, we did try hard, under Mr. Turner's chairmanship, to raise some money for these tests.‡ The suggestion has been made that every one who attends this convention should feel that he is a committee of one to go to his home town, his home industries and his home section, and try to support the drive for the funds to pay for these tests. It has been adopted as a policy that the Institute take no part of any of its own limited reserve to pay for tests, because any contribution it could spare from dues would be very small in the field of research which runs into large figures. The money should be contributed voluntarily, in large amounts and small by those who will benefit by the information to be gained. Those of us who have worked with Mr. Turner invite you all to get back of this drive for funds in the various cities and put a push behind your committee to get the total up to the necessary $25,000."

Contributions so far (April 1) amount to $12,077.69.

The contribution of two Universities —Illinois and Lehigh—is estimated at about $15,000, but this is in addition to the $25,000 estimated as the minimum of the fund to be raised.

The Portland Cement Association is contributing $6,000 in the time and expenses of two men, one under Professor Slater at Lehigh, one under Professor Richart at Illinois.

THE FUND TO DATE

Previously acknowledged		$10,702.50
Received since publication of March Journal		
Caye Construction Co., Brooklyn, N. Y.	$100.00	
Building Materials Dealers' Credit & Stat. Assn., St. Louis, Mo.	300.00	400.00
		$11,102.50

In addition to cash acknowledged above the Institute is glad to note contributions to the fund at a cash value:

Lehigh Portland Cement Co., Allentown. Cement, wood barrels and freight to University of Illinois and Lehigh University.	$743.55
Warner Co., Philadelphia—member National Sand and Gravel Assn., Inc. Sand and gravel for column tests shipped to Lehigh University.	231.64
Grand Total	$12,077.69

*The report will be found in the *Proceedings* pages of this JOURNAL.

‡New York City is already considerably over its original quota. In other cities the work is following more slowly.—EDITOR.

1. CONCRETE—INDISPENSABLE SERVANT

BY ARTHUR R. LORD*

As you ride o'er the roads in your automobiles
Have you thought of the concrete on which roll your wheels?
The servant of one and the servant of all,
Ever waiting your pleasure, your beck and your call,
It stretches away. In the distance the plain,
The foothills, the mountains, the ocean again;
New faces, new scenes, new hopes and new dreams;
Over each flowing highway the mystery gleams.
'Tis concrete that brings to man's spirit these springs
Of refreshment, and joy, and all other good things
That you find as you speed in your automobile
While the voice from the rear calls the turn of your wheel.

The light in the bulb, the power in the mills,
Tells of concrete in dams at the foot of the hills;
The house that is lighted, the wheels that are turned,
Are founded on concrete, on clinker that's burned
And ground to a fineness than flour more fine . .
The temple of business, the club where you dine
Are protected from fire (and the steel that would rust
Is saved from the water and oxygen's lust),
By concrete alone, while the bridges that rise
In arches most graceful, delighting the eyes—
How amazing the span!—the genius of man
Uses concrete to create on Nature's own plan.

II. AMERICAN CONCRETE INSTITUTE

So simple it seems . . . Mix cement with sand
And gravel (or limestone, if ready to hand)
Then stir in them water to make a fine paste;
(We use chiefly the stuff that other men waste!)
Put the "mud" in the molds and leave it to set,
And improve on the stone that the quarrymen get.
But ever remember that now the sequence
Includes technical knowledge and plain common sense.
Not hard work but head work, not careless but true
To the simple and rigorous laws that are new,
That spring from research, from experience grow,
And teach us the things it behooves us to know.

Forever we learn, ever forward we move;
Together we seek to step out of the groove.
In union is strength, in discussion is life,
Taken back to the job it reduces the strife.
In the friendly abrasion of mind upon mind,
We make it our purpose to leave none behind.
Our friendships are founded on work that we do;
Convention, committee-refresh them anew.
The thoughts and the facts we receive from our store,
Impose on each member the will to make more,
Give time and attention, strain sinew and nerve
Institute of American Concrete to serve.

III. THE BOARD OF DIRECTION

Little jobs, done with snap; bigger tasks with aplomb,
Afford us the clue, as the records we comb
To find, in the midst, the rare man who will work
(Consistently work, never thinking to shirk)
On the matters so pressing, both little and great,
Things that have to be done to establish the date;
Things needed for press and pressing for time.
Men steadily working, nor asking a dime;
Nor seeking advancement, but mounting each step
With glad recognition, demanding more pep
From oldsters and youngsters, whoever they be
That seek to advance in the same company.
For workers who work, are the Institute's need,
From shirkers who shirk, we hope to be freed.
For climbers who climb, (what a problem to solve!)
We at least should demand that some work it involve;
We should try to make known, ere the end of the race,
That climbers as workers have set a good pace!
We endeavor to pay, in the coin of our realm,
Both the man in the ranks and the man at the helm;
We can make his name known, his memory praised,
Wherever good concrete is sought or is raised;
We can train him to use his great gifts as a vector
To help other people; in short, a director!

*If an award is set up for versatility in the Institute—especially in these pages, contestants will be sorely handicapped by our new Vice-President. It is no everyday matter for an engineer to dally with the muse. With spring officially in our midst, and actually no doubt just around the corner, we realize that we are taking a long chance in opening this publication to odesters. But the rules and precedents have been searched for -hibitions, in and pro, and there is nothing of record. If this should become epidemic something will have to be done about it.—EDITOR.

BOUND VOLUMES 26 AND 27

SUFFICIENT extra *Proceedings* and *Abstracts* pages of each month's JOURNAL are printed to meet advance orders for bound volumes—ready after the last issue of Vol. 26 (June 1930) and for a limited number of extras. The non-member price for Vol. 26 is $12.50. The member price $6.00. Many members place a high value on their complete set of bound volumes (many of which are now out of print and unobtainable). Some members will have their JOURNALS bound. Others, to avoid possible loss, have ordered and paid for extra complete volumes for delivery when complete. Advance orders for such extra volumes received last year were taken at a very low special price to members—based on an estimate of actual cost. This special price was $3.00 to members and subscribers only and no more than two copies to each. Orders can now be accepted for Vol. 26 only at the non-member price of $12.50—member price $6.00. Orders for Vol. 27 (Sept. 1930 to June 1931) will be received from members and subscribers only until August 1, 1930 at a price not to exceed $3.50.* After that date the member price will be $6.00.

A TEST FOR SAND

A LETTER FROM F. O. FAREY†

ONE of the problems arising in this territory, is in the characteristics of sand for concrete purposes. While there is a great deal of literature published we beg to suggest a technical test be devised and standardized which would indicate whether or not the disintegration of shale and other rock had developed to an extent which would make it unsuitable for general concrete purposes.

A tentative suggestion is made that a definite quantity of sand be kept immersed in water for a definite time and then placed in an Abbe rotating ballgrinder and have the increase of silt and disintegrated matter determined. The amount of such disintegration could be standardized at a point considered as the maximum to be permitted.

The opinion of the writer is that ordinarily the amount of disintegration should be comparable with the disintegration which would naturally occur in the average concrete mixer when using sand which has been kept moist or wet for a considerable period of time, as sometimes occurs when using sand during wet seasons or taken from damp sections of bank sands or river sands.

*This price is based on payment in advance—the extra amount to be billed on receipt of order or as soon as a cost estimate can be arrived at, based on Vol. 26.

†Asst. general manager, Robert W. Hunt & Co., Ltd., Montreal.

INSTITUTE CANNOT MEET REQUESTS FOR ADVANCE COPIES OF PAPERS

REQUESTS are being received for copies of papers presented at the 26th annual convention and not yet published in the JOURNAL. It is impossible to meet these requests until papers are published. A number of such papers are available in manuscript, a few are still undergoing revision by their authors; some are in type for early publication. The secretary's office cannot undertake making typewritten copies nor ask the printer for unlimited galley proofs. Members will please be patient—they will be served *equally* when the papers appear in their JOURNAL.

STANDARDS OF DESIGN FOR CONCRETE*

A REVIEW

THE PREFACE indicates that this book of over 200 pages has been written by Lieut. Commander Ben Moreell, who acknowledges some assistance from Lieut. Joseph A. Wise (Univ. of Minnesota) and Mr. Harris Epstein (Bureau of Yards and Docks). It is printed by the Government Printing Office and is available through that office. The price is not fully determined but is expected to be fifty cents per copy.

The book includes directions, formulas, and diagrams for the design of all usual types of structural concrete; including buildings, docks, tanks, chimneys and retaining walls. It does not treat of arches nor dams, in detail. The most unusual feature is the completeness of the treatment. While the discussion is exceedingly compressed, and is directive rather than explanatory, many minor points are covered and rules of design given, such as are usually omitted in other writings of this kind.

In general, the recommendations of the Joint Committee and of the Joint Code are accepted—but there are many exceptions. No attempt is made to explain these departures from accepted standards in many instances. The use of new design diagrams to determine the shears and moments in continuous beams and rigid frames is a notable departure, fully explained. In this review it will be assumed that the reader is particularly interested in departures from and in subjects not covered by the report of the Joint Committee and the Joint Code.

*United States Navy Department, Bureau of Yards and Docks, No. 3 Y b, November 15, 1929.

In the selection of working stresses for design many departures will be found (without explanation). Some stresses are larger, some smaller than those given in the Joint Code. Values for tension in concrete and for unit stresses in combined torsional and direct shear are added. The spiral column is treated as a special case of the banded column.

The unit stress in combined bending and direct stress is lower when there is tension over part of the section than when the section is all in compression for spiral columns—*higher* for banded columns. (Table 1).

A formula for computing shrinkage and temperature reinforcement is given, which does not include any consideration (in the formula) of size of bar or of bond resistances. A plain bar is just as good as a deformed bar or as wire mesh by this formula. A preference for small deformed bars is stated.

As noted above the usual arbitrary moment coefficients are discarded, and diagrams substituted, in the design of continuous beams and rigid frames.* These diagrams take account of varying degrees of restraint and include common concentrated loadings as well as uniform loading. The effect of E_c, the stiffness factor of the various qualities of concrete that may be used in buildings, is neglected in this discussion. Wall support as well as column support and the restraint of lintel beams of various sizes and also the effect of haunches and fillets are analyzed.

The case of the two-way slab supported on all four edges is treated, following the paper by Westergaard (Proceedings Am. Conc. Inst. 1926.)

*Cf. Stiteman diagrams in JOURNAL AM. CONCRETE INST., Jan. 1930, and the Cross paper, JOURNAL AM. CONC. INST., Dec. 1929, which treat of the same matter.

These coefficients are based on freely supported slabs, but are here applied to slabs which are recommended to be built monolithically with their supports.† The effect of concentrated wheel loads is treated in an unusual manner and with unusual completeness.

The discussion of flat slabs introduces much that is new (and adds to the labor of design) by considerations of torsion and gives formulas for torsional stresses and design. The allowance for shearing resistances ignores the distribution of the slab reinforcement. The treatment of openings through the floor is discussed and rules given. A special bond formula for flat slabs is stated.

Much attention is given to the important matter of bending in columns. In the case of concrete piles, not only the final load condition, but also the handling and driving stresses are analyzed.

The treatment of tanks and reservoirs is exceptionally complete, including several design diagrams. All parts—side walls, floors and roofs, circular and rectangular tanks, and many special details are covered. Under the design of chimneys, a treatment of earthquake stresses is included. Design charts are given for stresses in the stack due to deadweight, wind and temperature, considered separately and combined. The design of a stack footing on soil or on piles is facilitated by other diagrams.

The deflection of concrete beams is analyzed after the manner of Turneaure and Maurer. A selected and concise test of references to works consulted by the author is appended. Another appendix contains influence lines for continuous beams of equal span and uniform moment of inertia. And finally, but not least, there is an index.

The reviewer does not wish to influence the readers' judgment. There are two ways to prepare a set of design standards. A committee will report a compromise result, presumably better than any member alone would have produced, because of the criticism of all members. An individual will report his own thoughts and practices—he cannot be outvoted even if he submits to criticism. This book is an individual's work. It has all of the strength and some of the weaknesses that necessarily result from that method. Every designer has to make decisions on matters of design in which adequate data are still lacking or not yet published. But such decisions are rarely published. Many such matters are reported in this book. It will do any designer a world of good to study—not merely read—this book, with its many departures from so-called standard practice. It will cost far more in time than in money, but both time and money will be well expended. And when you have read and reflected and computed and passed judgment, this reviewer would like to know your conclusion. And I am sure that Commander Moreell would like to receive a copy of your letter also.—ARTHUR R. LORD.

†Cf. The Schuster design coefficients used in Chicago and elsewhere.

The Directory for 1930 is at last on the press—delayed for committee announcements—and will be mailed a few days following this JOURNAL.

CONCRETE MASONRY

THERE was a time when two questions were asked about masonry units —their strength, their absorption. The Advisory committee in planning committee work in connection with concrete block and building tile has many more things to consider and many new jobs for the Board of Direction to assign. Here for instance is just one paragraph in the rapidly swelling Advisory Committee files:

The essential requirements of conconcrete masonry are nailability, cutability, plaster bond, sound insulation, heat insulation at low temperatures, fire resistance, watertightness, effective mortar bond, light weight, ease of setting and economy. These requirements are affected by cement content, proportions and grading of aggregate, nature of aggregate, method of manufacture, degree of curing, etc. High cement content will improve fire resistance and increase watertightness, but it may interfere with plaster bond, cutability, nailability, etc. One design of unit may be more easily cut than another but it may also have poorer fire resistance for a given weight of material. An analytical study of concrete masonry, including the design of the units, should be made in order to establish the best combination of these requirements and the best methods for producing the desirable combination.

CONCRETE SHIPS

ONE of the interesting things viewed by some of the Institute conventioners at New Orleans was a group of three concrete ships, among them the Faith. A. E. Lindau, acting as chairman of one of the convention sessions called attention to them thus:

"I would like to also announce to those who may be interested, that they can see three reinforced concrete ships within a short distance of the hotel, if they are interested in doing so. Your chairman and some other members of the Institute were out to see them today. The good ship Faith, which I think was the first one to be launched, is anchored in the river, and alongside of the Faith are two reinforced concrete tankers. The examination we have made so far indicates that the concrete, particularly in the tankers, which was of lightweight aggregates, is in very excellent condition; in fact, there is a piece of it here. Some one asked me it it had begun to break up. It has not. The ship suffered a collision, only the railing being damaged, and I was anxious to see what the concrete looked like. The reinforcement is exposed in places, but in spite of the exposure which this reinforcement had, it was very bright on the surface."

There are very few cramping formalities about the *News Letter* section of the JOURNAL. It is for the use of members in furthering the interests of the Institute. If you have an idea why not pass it on. Please do not wait to be invited. Even a new and imperfect idea has a way of growing in value sometimes when exposed to the reactions of others similarly interested—EDITOR

WILSHIRE PROFESSIONAL BUILDING, LOS ANGELES

ARCHITECTURAL TREATMENT WITH CONCRETE

What follows was not written as a formal contribution to these pages but developed piece-meal in correspondence—excerpts from letters by Mr. Mayo, here published by permission as of interest to those who look to good workmanship to make concrete a highly acceptable architectural medium. Mr. Mayo is president of Luther T. Mayo, Inc. (engineering, financing, building), Los Angeles. —EDITOR.

BY LUTHER T. MAYO

REGARDING the use of concrete in obtaining desirable architectural effects, we believe the particular methods employed by us in the erection of concrete buildings in the Southern California district have considerable merit and value, where the architectural treatment has been accomplished with exposed concrete.

The desirability of concrete for buildings of permanency and stability is obvious, but properly to characterize the structure with fitting and lasting ornamentation has previously resulted, in most cases, in the use of other materials applied over the rough concrete, which has caused much expenditure and loss of time, and invariably unsatisfactory results. The principal advantage in adopting lined forms and waste molds is to be able to produce an artistic concrete building monolithically placed in its entirety. Another advantage enables the general contractor to schedule the work, thereby retaining full control of the progress of

AMERICAN STORAGE BUILDING, LOS ANGELES

the erection, and eliminating all possible chance of expensive delays.

Blank walls or panels, when erected and placed in lined forms are given life and character, while cornices, bases, capitals, etc., that make up the ornamentation in general, when waste molds are used, will bring out the same artistry of design produced by other materials (unless, of course, that design is too intricate) and in our experience, fully satisfactory to the architect. Additional care is required to obtain well placed concrete, but the resulting saving in time and expense is well worth the effort.

We find that even more beauty and architectural excellence can be obtained with these methods, as evidenced by some of our recent structures—such as: the American Storage building, the Chateau Elysee apartments, the Santa Monica Professional building, and the Wilshire Professional building—all of which are prominent examples of the highest type of concrete construction.

In lining forms we use, inside the lumber forms, a composition board, preferably having a smooth surface and capable of withstanding water absorp-

Santa Monica Professional Building, Santa Monica

tion and wet concrete for three or more uses. We have used Masonite products satisfactorily for this purpose in the past, but have in all applications carefully oiled all contact surfaces to insure easy stripping, and to aid in prolonging the life of the board.

The actual details of form assembly and erection would necessarily vary with individual experiences and job conditions. The customary forms are usually built in panels, using common boards and studs. The form lining, as previously stated, is placed inside of that, and stone, or other marking, of whatever dimensions desired, is obtained by planting strips on the face of the lining. This is then ready for erection.

Additional care must be taken in pouring, both as to consistency of mix and tamping, to prevent voids—also, stripping must be accomplished in such a manner as to leave the surface without unnecessary blemishes. Careful oiling of contact surfaces, between pourings, is required to do this, and offset the suction of the concrete. There is no special technique, except careful supervision in erecting, stripping, and pouring into these forms. Spacing, alignment, solidity of forms, elimination of protruding seams between floors and other well known precautions to be anticipated, are all a part of every good contractor's knowledge. No set rule or practice can be established for this procedure. It depends on the plans and workmanship of the architect and builder.

Coloring of finished surfaces is also a matter of choice—several different opportunities being available by acid staining, and concrete paint, or even allowing the concrete to remain untouched lends a tendency to stone effects.

IN CONSTANT REPAIR

(Conversation with Dr. Samuel Johnson)

Dr. Johnson: A man, Sir, must keep his Friendships in a state of Constant Repair.

The Author: And no less, Sir, his Information.

2. ON FINE MESHES

BY ARTHUR R. LORD

IT HAS been said that a minority is always right and it is unquestionably true upon occasion. The first man to discover a new truth is surely for some time in the minority, until others have investigated and accepted the new in the place of the old belief. In the discussion of concrete design we have had, at times, a vociferous and unquenchable minority—a minority that persists uncorrected in the error (or truth) of its ways regardless of repeated defeats at the committee or convention polls. Is it possible that the majority—the code writers and the standardizers—have overlooked or undervalued some truths that the fighting minority have rightly held important? It seems quite possible to me that if we should re-stage some of the exciting convention "scraps" of fifteen and twenty years ago with the same combatants, the majority would accord the minority a more considerate hearing, and might even grant the soundness in part, if not in their complete application, of some of the rejected contentions. Are we losing our fighting minority? It seems a long time since attacking contestants clamored for the floor and the besieged, overwhelmed by the numerity of the attack, remembered departing trains or otherwise protected their verbal rout! Those were the good old days when the young giant, Concrete, was shooting from youth into young manhood.

In the comparative quiet of present days we may do well to reconsider some of the old turning points of opinion, to see if they may not direct our designs to good advantage and economy even though time has proved them unnecessary limitations on *safe* design. These thoughts have been stirred by a recent committee meeting in which, after many years of silence, I heard again one of the perennial champions of ancient causes defend some of these almost forgotten tenets. It happened to be the old battle ground of the effect of spiral reinforcement on column strength. To him, now as ever, the yield point of a column as shown in a test—the point at which the concrete gives way and begins to throw considerable pressure against the spiral—signifies nothing. Only the ultimate test load is important in determining the safe working load on the column. In our new reinforced concrete column investigation we shall doubtless feed fresh fuel to that fire. But in the restatement of this honored fallacy, the old time warrior suggested details that we sometimes forget. He insisted that the spiral wires should be spaced much closer together than our usual limit of three inches; especially so if the intensity of load on the column is high. He asserted as a proven truth that small spiral wire closely spaced is a *far more effective* lateral reinforcement than the same weight of wire of a larger diameter and spaced so far apart as three inches. And he was no less emphatic that only a high percentage of spiral should be shown when concrete of high compressive strength is used in the design. Leaving aside the greater battle and

considering only these marginal combats, is this a case in which the minority is right? Do we have tests that decide such important minutia of design and indicate aright the validity and significance of this warrior's contentions?

We have, perhaps, a parallel case in the design of stirrups. What a battle ground that was! Tests have surely demonstrated that the majority was safe—but not that they were "right"—at least not a hundred per cent right. Vertical stirrups do take tensile stress, despite the minority's death stand on the principle that they cannot. But we have to admit that vertical stirrups are less effective agents in resisting diagonal tension than are inclined bars or stirrups. We know that the stress in vertical stirrups is very small until the concrete of the web cracks in tension—as is true also of the stress in the main longitudinal reinforcement in a beam. But there *is* a difference—a marked difference in degree and possibly in kind between the two cases. So far the minority was "right." But here again, it is a minor matter that now concerns me rather than this old major issue that we may now regard as settled—the vertical stirrup is all right but an inclined one is to be preferred! The minor matter again concerns the distribution of the steel among the stirrups. We were just starting an investigation of this matter at Bethlehem for the Emergency Fleet Corporation when our work was called off. Had the Germans but held out another six months we might have had a conclusive answer! The few tests that were made, however, did indicate that here again stirrups made of smaller bars more closely spaced were much more effective than the same weight of steel in larger bars at greater spacing. It seemed to indicate a very rapid loss in efficiency as the spacing was increased beyond certain limits which are set up in our standards. But even within the accepted limits there were marked variations. Has more light been thrown on this matter in tests that have escaped my attention or possibly not yet found publication, made in these ten post-war years? It seems certain that our concentration on the water-cement ratio has pulled most of the research miners out of the reinforced concrete pits and has deprived us of much valuable information about reinforced concrete design. I refer to its refinements rather than to matters of safety. But surely *economy* is involved and is a matter worthy of further research and discussion. In this day when welded shear units are finding a ready sale to contractors as a substitute for stirrups shown in the design, it is no longer impractical to require small stirrups at close spacings.

In still a third field—and here it seems probable that the Institute may soon be provided with funds with which to put forward an adequate research—we find this same minor point illustrated. Again I use the word "minor" as relating to safety—it may be quite important as to economy. In slabs—short span slabs, as we generally speak of them—it appears to be true (a militant minority assert it to be true), that a welded wire mesh in which the steel is in the form of small wires rather closely spaced is *much more effective* than the same weight of steel in bars at greater spacing. There may be other matters involved here to obscure this issue. Not only natural science but also commercial interest—established trade practice—is involved to a high degree. After all is known, where will the balance fall between the relatively

cheap ¼-in. round temperature and shrinkage bar (a waste product from another use of steel and hence a profitable item for steel jobbers) and the more expensive welded wire mesh?

In all these aspects of the advantages of a fine mesh over a coarse mesh, I should welcome the assistance of Institute thinkers and doers in keeping our facts and theories in a state of constant repair.

TRAFFIC CONTROL

A LETTER FROM E. N. GOLDSTINE*

IN THE *News Letter* for February 1930 Mr. Lord prompts discussion of the matter of control of concrete operations.

In the last ten years I have had the opportunity as a superintendent of construction to observe concreting operations in various parts of the country —Indiana, Florida, New York, California, and Pennsylvania. For the most part, the jobs have ranged in cost between $100,000 and $400,000, and might be considered typical of the average medium sized construction project. My experience leads me to believe that specifications should very clearly cover certain phases of proportioning, mixing, and placement of concrete. The right kind of specifications and their rigid enforcement would unquestionably result in producing better concrete, and would have a tendency to drive out of the construction industry irresponsible contractors and incompetent or dishonest workmen.

I am strongly in favor of batchmeters that will not permit the discharge of concrete materials until they have remained in the mixer drum the time required by the specifications. The majority of construction superintendents will run the materials through the mixer as fast as they can take it away, with the net result of concrete insufficiently mixed. Yet the cost records of the foreman who is satisfied with 20 second mixing prove him to be the superior of the man who insists on a full minute mix—in the eyes of most contractors. And the contractors who want to do good work must compete with those who do not know and do not care.†

Accurate water measuring devices are beginning to get wider recognition, but are not as universal as might be desired. Inundators might be specified on the larger jobs. Where wheelbarrows are used to measure aggregates, they should be of a type similar to the Sterling measuring barrow, so that the quantity of material obtained is not open to doubt. Incompetency is often as important a factor in improper measurement of aggregates as the desire to skimp materials.

I have noticed that most of the more experienced men have their own ideas on concrete which are not altered by anything short of the threat of discharge. Foremen and finishers insist upon excess water when it is not required for proper manipulation. The other extreme is often presented by inspectors who expect thin wall sections to be poured with concrete having a slump of 2 in. and an insufficient amount of sand.

As I see it, the prime need in concrete operations is a complete understanding of what needs to be done, and then an honest desire to attain a reasonable standard of competent performance. Altogether too often the desire to save money on materials and labor out-

*San Francisco.

†Here is a bold challenge of the good faith or intelligence of contractors, superintendents and foremen. Perhaps somebody will speak up.—EDITOR.

weighs the necessity of complete compliance with intelligent specifications, and the owner gets an inferior building, but seldom knows the difference.

COMMITTEES

FEB. 1930—FEB. 1931

ADMINISTRATIVE committees for the current year are:

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(Acting for the Board between Board Meetings)

Chairman—D. A. Abrams
Secretary—Harvey Whipple

E. D. Boyer
M. M. Upson
S. C. Hollister

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A. W. Stephens (Chairman Dept. 300, Engineering Design)

Rexford Newcomb (Chairman Dept. 400, Architectural Design)

A. B. Cohen (Chairman Dept. 500, Specifications)

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J. C. Pearson
A. N. Talbot
Benjamin Wilk
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Technical committees authorized by the Board since February 1929 will be listed next month as continued for the current year. (All committee assignments are for the current convention year—February to February. Committees are discharged when their specific tasks are completed or when sufficient progress to justify continuation is not evident). A number of new committees are authorized and will be announced when a necessary nucleus of personnel is found for each assignment.

The price of $1.25 per copy of this JOURNAL is a non-member price. Members may buy extra copies at 50 cents each.

NEW MEMBERS

TWENTY-EIGHT applications for membership in February 1930 have been approved by the Board of Direction, from the following:

Allen, George F., P. O. Box 1031, Sta. "C", Los Angeles, Calif.

Art Stone Co., 2300 East Third St., Sioux City, Iowa. (Carl J. Ludwig, Gen. Mgr.)

Chicago Architectural Stone Co., 1743 N. Kenton Ave., Chicago, Ill. (M. A. Williams, Vice Pres.)

Cochran-Tefft Inc., Shreveport, La. (R. H. Tefft)

Eighmie, Borden C., R. F. D. No. 4, Schenectady, N. Y.

Ferguson, Phil. M., 711 West 24th St., Austin, Texas.

Glen, Michael G., Division of Highways, San Bernardino, Calif.

Gottschalk, Otto, 137 Smith St., Perth Amboy, N. J.

Grube, L. E., 1227 N. 8th St., Sheboygan, Wisc.

Habley, C. G., 403 Memorial Dr., Cambridge, Mass.

Halliburton Oil Well Cementing Co., Duncan, Oklahoma. (Hayden Roberts, Chemist.)

Infante, Percival M., New York Central R. R. Lines, 406 Lexington Ave., New York, N. Y.

Jackson, Paul E., 12 May St., Worcester, Mass.

Jachncke, Walter F., 814 Howard Ave. New Orleans, La.

Johnson, O. K., 1000 Franklin Ave., Waco, Texas. (J. E. Johnson Const. Co.)

Larsen Engineering Co., Merchant's National Bldg., Omaha, Nebr. (C. H. Larsen, Pres.)

Madden, E. Pencle, 1228 N. Rendow St., New Orleans, La.

Moreell, Ben, Bureau of Yards & Docks—Navy Dept., Washington, D. C.

Paxson, G. S., State Highway Dept., Salem, Oregon.

Phillips, R. S., 33 W. Grand Ave., Chicago, Ill.

Reynolds, C. W., Parliament Wharf, 165 Grosvenor Road, Westminster, London, England.

Rhodes, J. Morton, P. O. Box 776, Freeport, Texas.

Sargent & Lundy, Inc., 20 N. Wacker Drive, Chicago, Ill. (J. C. Sanderson)

Saunders, W. D., 251 Kearny St., San Francisco, Calif.

Thomas & Sons Co., The R., Lisbon, Ohio. (H. P. Sleeman)

Votaw, Curtis H., 212 S. Burns, Holdinville, Okla.

Wetzel, E. H., c/o Nucarth Stone Co., Carthage, Mo.

Zenk, Paul H., 1137 Dueber Ave. S. W., Canton, Ohio.

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THE COLORATION OF CONCRETE 616
Report of Committee 703, RAYMOND WILSON, Author-Chairman

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Discussion of the contributions in the following 116 pages is scheduled for publication in the JOURNAL *for October 1930 (except as otherwise noted) and should be available not later than September 1.*

Reinforced Concrete Column Investigation

*Progress Report of Committee 105**

W. A. SLATER, CHAIRMAN

Statements of the purpose in starting an extensive investigation of reinforced concrete columns have already appeared in the Journal† of the American Concrete Institute for November, 1929 and January, 1930. The present report aims to give something of the progress of the work.

As a first step, a survey is being made of the existing data on tests of reinforced columns. A list of the investigations which have been carried out in this country and Canada is given in Table 1; a similar list of foreign tests in Table 2. The latter is not complete as yet, and the former may not be. It would be appreciated if anyone knowing of test results not included in these lists would report the references to Committee 105.

The references are limited to tests of columns of the type included in the committee's program, that is, to concrete columns with longitudinal or lateral reinforcement or both. Steel columns with concrete reinforcement are not at present included in the investigation nor in Tables 1 and 2.

No attempt is made to abstract fully in this report, the results of the investigations listed. However, attention is called to a few of the outstanding features. The conclusion that the yield-point strength of spirally reinforced concrete columns was represented by the sum of the yield-point strength of the longitudinal reinforcement and the ultimate strength of the plain concrete was reached at nearly the same time by Gillespie and Withey in America and by Mörsch in Germany. Talbot recognized that the yield-point strength of the columns with only lateral reinforcement was represented by the ultimate strength of the plain

*Presented at the 26th Annual Convention, February 12, 1930.
†*News Letter* Section.

TABLE 1—TESTS ON PLAIN AND REINFORCED CONCRETE COLUMNS IN U. S. A. AND CANADA

Columns Tested By	Report Published		Column Dimensions		Reinforcement			No. of Columns Included In Report	Test Made At
	When	Where	Cross Section	Length Feet	Vertical Per Cent	Horizontal Kind	Horizontal Per Cent		
J. E. Howard	1897	Tests of Metals	12″ Square	2 to 14	0 to 1.57		0	48	(1)
Gaetano Lanza	1903	Transactions, Am. Soc. C. E.	8″ Square 10″ Square	5 to 17	0 to 6.25		0	28	(2)
J. E. Howard	1905	Tests of Metals	12″ Square 11″ to 13½″ Dia.	8	0 to 4.63	Exp. metal Steel lath.		37	(1)
E. S. McCaustland	1905	"Engineering News"	10″ Dia.	3⅓	0 to 2.55	Hoops	0 to 2.0	13	(3)
J. E. Howard	1906	Tests of Metals	10″ to 12″ Dia. 12½″ Square 9″ Square	8	0 to 1.53	Hoops wire Cage		41	(1)
A. N. Talbot	1907	Bull. 10, Univ. of Illinois	12″ Square 9″ Dia.	6, 9, 12	0 to 1.5	Tie Rods Spirals		18	(4)
A. N. Talbot	1907	Bull. 20, Univ. of Illinois	12″ Dia.	10	0	Hoops	0 to 3.2	51	(4)
J. G. Houghton W. P. Cowles P. Gillespie	1908	"Engineering News"	9″ Square	9½	0 to 2.43	Hoops		17	(5)
G. W. Swan	1908	"Canadian Engineer"	3½″ x 5″	1⅔ to 7	0 to 2.5	Hoops Spirals Spirals		7	(6)
M. O. Withey	1909	Bull. 300, Univ. of Wisconsin	Various	10	0 to 4.5	Hoops	0 to 2.0	20	(7)
P. Gillespie	1910	"Canadian Engineer"	6″ Dia.	1¾	0 to 4.42	Hoops	0 to 5.7	25	(6)
H. C. Berry	1910	"Engineering Record"	14″ Square	12	0 to 2.4	Spirals	0 to 1.75	9	(8)
M. O. Withey	1911	Bull. 466, Univ. of Wisconsin	10″ Dia.	8½	0 to 10.0	Spirals	0 to 2.0	74	(7)
Am. Con. Institute represented by A. N. Talbot	1915	Proceedings, Am. Concr. Inst.	21″ Dia.	9⅓ Snaft	0 to 6.0	Spirals	0 to 2.0	24	(9)
C. G. Wrentmore, Brodie and Grey	1915	Transactions, Am. Soc. C. E.	4″ Dia.	2¼	0 to 4.10	Spirals		104	(10)
F. P. McKibben, A. S. Merrill	1916	Proceedings, Am. Concr. Inst.	14″ Dia.	5 to 20	0 to 4.0	Spirals	0 to .96	33	(11)
F. E. Richart, Brandtzaeg, Brown	929	Bull. 190, Univ. of Illinois	10″ Dia.	3⅓	0	Spirals	0 to 4.41	23	(4)
							Total	572	

(1) Watertown Arsenal
(2) Massachusetts Institute of Technology
(3) Cornell University
(4) University of Illinois
(5) Department of Buildings, Minneapolis, Minn.
(6) University of Toronto
(7) University of Wisconsin
(8) University of Pennsylvania
(9) Bureau of Standards, Laboratory at Pittsburgh
(10) University of Michigan
(11) Lehigh University

TABLE 2—LIST OF TESTS ON PLAIN AND REINFORCED COLUMNS IN EUROPE

Columns Tested By	Report Published		Column Dimensions		Reinforcement			No. of Columns Reported	Tests Made At
					Vertical Maximum Per Cent	Horizontal			
	When	Where	Cross Section Inches	Max. Length Feet		Kind	Per Cent		
C. Bach	1905	(5) Heft 29	Octagon	3.28	2.85	Spirals	0 to 7.05	61	(8)
A. Considére	1907	(1)	Various	1.6 to 16.4	3.97	Spirals	0 to 2.04	53	(7)
F. Emperger	1908	(2) Heft 8	7.1 Sq.	3.28 to 11.8	29.2	Hoops	Various	37	(8)
M. R. vonThullie	1909	(2) Heft 10	3.15 Sq.	4.95	2.78	Hoops	Various	485	(9)
M. Rudeloff	1910	(3) Heft 5	Various	2.95 & 6.56	.9	Hoops	Various	97	(10)
M. Rudeloff	1912	(3) Heft 21	11.8 Sq.	3.28 to 6.56	3.15	Hoops	Various	45	(10)
A. Kleinlogel	1912	(2) Heft 19	Octagon	3.28	2.0	Spirals	0 to 3.6	24	(11)
C. Bach	1913	(4) No. 50	12.6 Sq.	3.95 & 29.5	2.8	Spirals	Various	27	(8)
C. Bach—O. Graf	1914	(5) Heft 166–169	15.75 Sq.	6.56 & 8.2	1.9	Hoops Spirals Hoops	Various	61	(8)
M. Rudeloff	1914	(3) Heft 29	Various	4.25	1.4	Spirals	Various	68	(10)
C. Bach	1915	(4) No. 44	Octagon	1.69	.7	Spirals	Various	30	(8)
M. Rudeloff	1915	(3) Heft 34	11.8 Sq.	4.25		Hoops	Various	12	(10)
Otto Graf	1921	(6) 3rd Ed.	Various	2.95 & 3.95	2.8	Hoops Spirals	Various	27	(8)
							Total—	1027	

(1) Commission du Ciment Arme
(2) Forscherarbeiten auf dem Gebiete des Eisenbetons
(3) Deutscher Ausschuss für Eisenbeton
(4) Zeitschrift des Vereins Deutscher Ingenieurs
(5) Forschungsarbeiten auf dem Gebiete des Ingenieurwesens
(6) Handbuch für Eisenbetonbau
(7) National Road and Bridge Laboratory
(8) Technical Institute at Stuttgart
(9) Technical Institute at Lemberg
(10) Royal Prussian Materials Testing Laboratory
(11) Technical Institute at Dresden

concrete. This is in harmony with the conclusion formerly pointed out, since there was no longitudinal reinforcement in this series of Talbot's columns. He had previously recognized a somewhat similar relation for longitudinally reinforced columns with spirals, but did not assign the same strengths to the concrete in the reinforced as in the plain columns. Instead, he used the difference between the ultimate strength of the column and the yield-point strength of the steel as a measure of the strength of the concrete in the reinforced column. References to the data supporting these conclusions are given in Tables 1 and 2.

The significant fact is that these four investigators recognized that the lateral reinforcement had no function in adding to the yield-point strength of the column. However, although their statements of findings are couched in varying terms their conclusions agree in attributing to the spiral an addition to the ultimate strength of the column in an amount at least 2 or 3 times as great as would be added by an equal volume of longitudinal reinforcement. This is in agreement, also, with Considere's earlier work from which the conclusion was drawn that lateral reinforcement is 2.4 times as effective as an equal volume of longitudinal reinforcement.

All the investigators apparently agree as to the usefulness of the lateral reinforcement in practice in adding toughness to the structure and in increasing the factor of safety. They disagree, however, regarding the manner in which the spiral reinforcement may be utilized in structural design.

It is the aim of this committee to investigate the actual usefulness of the strength added by the lateral reinforcement (as distinguished from toughness), in determining working loads for columns. The program of tests includes 573 columns, strikingly close to the total number (572) represented in Table 1 as having been tested to date in this country and Canada.

For the purpose of insuring uniformity of the cement to be used in this investigation, the Lehigh Portland Cement Co., which is donating the cement for both laboratories, set aside 225 barrels from a single bin of cement for this purpose. During the packing of these barrels, test samples were taken from barrels No. 25, 75, 125, 175, and 225. These five barrels are being retained by the cement company for sampling and testing at later times during

TABLE 3—RESULTS OF STANDARD TESTS FOR CEMENT; TESTED BY LEHIGH PORTLAND CEMENT COMPANY

	Cement From Barrel No.				
	25	75	125	175	225
Water for normal consistency per cent by weight	22.0%	22.0%	22.0%	22.0%	22.0%
Time of initial set, hrs. min.	4:05	4:05	3:55	3:50	3:45
Time of final set, hrs. min.	7:00	6:50	6:50	6:40	6:30
Soundness	O. K.	O. K.	O. K.	O. K.	O. K.
Fineness per cent (passing No. 200 sieve)	88.2%	88.5%	89.0%	89.6%	89.3%
Tensile Strength of Standard Mortar Briquettes in Lb. Per Sq. In.					
Age: 1 day	115	140	130	115	125
	120	140	145	135	125
	115	125	140	120	115
Average—	117	135	138	123	122
3 days	250	275	270	260	260
	245	255	255	240	255
	230	250	240	245	235
Average—	242	260	255	248	250
7 days	305	355	370	325	320
	320	330	345	345	315
	330	350	345	325	340
Average—	318	345	353	332	325
28 days	440	420	420	475	450
	465	460	510	470	470
	440	475	495	465	450
Average—	448	452	475	470	457

the period of making the columns. The other 220 barrels have been shipped to the University of Illinois and Lehigh University. This cement is packed in paper lined wooden barrels and is stored in a dry place so that there should be little deterioration during the period of making the columns.

From the samples taken during the packing of the barrels, the Lehigh Portland Cement Co. made standard cement tests (Table 3) and tests on 3 by 6-in. cylinders of 1 : 1.9 : 2.8 concrete with enough water to give a water-cement ratio of 0.84 after having

TABLE 4—COMPRESSIVE STRENGTHS OF 3 BY 6-IN. CYLINDERS; TESTED BY LEHIGH PORTLAND CEMENT CO.

		Cement Taken From Barrel No.				
		25	75	125	175	225
Average flow (increase in diameter %)		73	67	70	75	71
Average water-cement ratio (nominal)		.909	.910	.909	.909	.907
Average water-cement ratio (net)		.836	.837	.836	.836	.834
Compressive Strength—Lb. Per Sq. In.						
Age: 1 day		409	426	402	406	404
		363	325	417	397	400
		375	369	379	360	402
	Average—	382	377	399	388	402
2 days		855	946	846	815	870
		801	795	833	832	828
		809	742	830	862	789
	Average—	822	828	836	836	829
4 days		1470	1550	1590	1550	1510
		1400	1360	1470	1440	1530
		1440	1380	1470	1550	1430
	Average—	1440	1430	1510	1510	1490
7 days		2000	1860	2040	1990	2030
		2170	2220	2140	2200	2140
		2250	2250	2230	2410	2110
	Average—	2140	2110	2140	2200	2090
14 days		3660	3540	3690	3480	3500
		3260	3220	3270	3250	3330
		3400	3590	3560	3570	3550
	Average—	3440	3450	3510	3430	3460
28 days		4670	4530	4650	4450	4200
		4200	4270	4730	4290	4450
		4410	4610	4640	4690	4810
	Average—	4430	4470	4670	4480	4490

allowed for absorption by the aggregate. One set of three specimens was made for each of the 5 bbl. of the lot of cement set aside for this purpose. The tests were made at ages of 1, 2, 4, 7, 14,

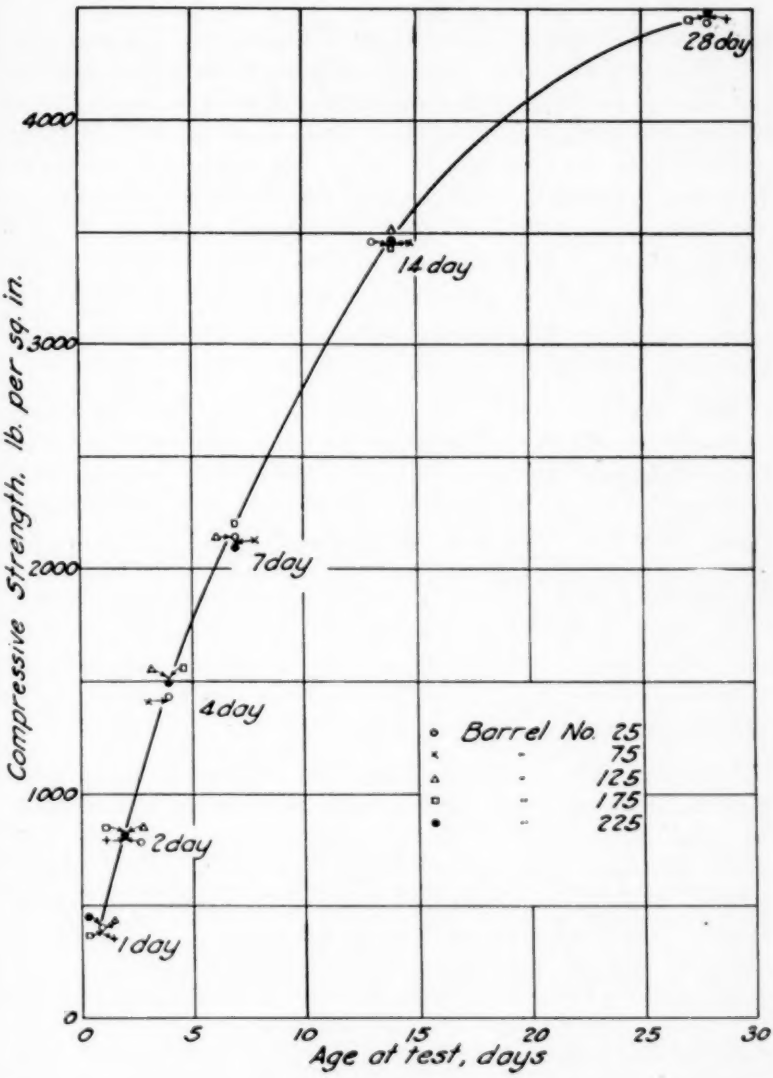

FIG. 1—CONTROL TESTS OF 3 x 6 IN. CONCRETE SPECIMENS BY LEHIGH PORTLAND CEMENT CO.

and 28 days. The individual compressive strengths of the concrete for these ages are given in Table 4 and the averages in Fig. 1. The extreme range in 7-day strength was from 2090 to 2200 lb. per sq. in., and the average strength was 2160 lb. per sq. in. The corresponding range for the 28-day tests was from 4430 to 4670, with 4510 as the average. In Table 4 it will be seen that the variation in strength from barrel to barrel was no greater than the variation for individual specimens from the same barrel. The 28-day strength was 65% greater than that given by the equation $S = \frac{14000}{7^x}$ where S is the compressive strength and χ is the water-cement ratio. It is evident that the cement used is of good quality as regards uniformity and strength.

Also a series of tests has been carried out at Lehigh University for the purpose of determing the proportions necessary to give the required strength and workability with the aggregates which have been secured for this investigation. The aggregates, both fine and coarse, have been furnished free of charge by the Warner Company of Philadelphia, member of the National Sand and Gravel Association. Both the sand and gravel came from near Morrisville, Pennsylvania. The sand has 9% passing the No. 48 sieve and 50% between the No. 48 and No. 28 sieves. Its average fineness modulus is 2.75. The maximum size of the coarse aggregate is ¾ inch.

A series of tests was made using three different proportions of sand to total aggregate. These proportions were 35, 40, and 45% by weight. For each of these groups four mixes (cement and mixed aggregate) were used, 1 : 1.5, 1 : 4, 1 : 6, and 1 : 8 by weight. For each group the water content ranged from about 3.5 to 10 gal. per sack of cement, thus securing the necessary range in strength. All the specimens in this series were tested at an age of 28 days.

The purpose was to secure water-cement-strength curves with some range of workability for each mix, so that from the resulting data a mix of the required workability for each strength might be selected for making the columns which are to be tested at 28 days. For a given water-cement ratio the range in workability was secured by the variation in the sand-aggregate ratio.

TABLE 5—28 DAY STRENGTHS OF CONCRETE FROM PRELIMINARY TESTS AT LEHIGH UNIVERSITY

Ref. No.	Mix By Weight	Water Content Gal./Sack	Average Slump Inches	Cement Content Sacks/Yd³	Sand Per Cent of Total Aggregate	Strengths At 28 Days Individuals			Average Lb./In²	Maximum Variation From Average, Per Cent
1	1-.53-.97	3½	2¼	14.8	35	7420	7190	6930	7180	3.5
2	1-.53-.97	4½	9	13.8	35	6140	5680	5780	5870	4.6
3	1-1.4-2.6	5½	6	7.6	35	5160	5400	4930	5160	4.6
4	1-1.4-2.6	6½	9¼	7.3	35	4080	4140	4260	4160	2.4
5	1-2.1-3.9	7	3½	5.5	35	2990	3640	3720	3450	13.3
6	1-2.1-3.9	8	7½	5.4	35	2690	2640	2690	2670	1.1
7	1-2.8-5.2	9	3½	4.3	35	1790	2080	1850	1910	8.9
8	1-2.8-5.2	10	5	4.2	35	1350	1610	1430	1460	10.8
9	1-.60-.90	3½	2¼	14.8	40	5710	6590	7570	6620	14.3
10	1-.60-.90	4½	9½	13.8	40	5350	5000	6030	5460	10.4
11	1-1.6-2.4	6	8½	7.5	40	4760	4850	4930	4850	1.9
12	1-1.6-2.4	5	1½	7.8	40	5580	5680	6130	5800	5.7
13	1-2.4-3.6	7½	5½	5.4	40	2920	2870	3300	3030	8.9
14	1-2.4-3.6	6½	1	5.6	40	4020	3910	4060	4000	2.2
15	1-3.2-4.8	10	4	4.2	40	1410	1490	1610	1500	7.3
16	1-3.2-4.8	9	1	4.3	40	1990	2040	1930	1990	3.0
17	1-.68-.82	4	7½	14.3	45	6680	6400	5560	6210	10.4
18	1-.68-.82	3½	2¼	14.8	45	7120	7230	7980	7440	7.3
19	1-1.8-2.2	6½	9	7.3	45	4340	3650	4380	4120	11.4
20	1-1.8-2.2	5½	4¼	7.6	45	4810	5100	5440	5120	6.2
21	1-2.7-3.3	8	6	5.4	45	2570	2720	2790	2690	4.5
22	1-2.7-3.3	7	1¼	5.5	45	3400	3270	3620	3430	5.5
23	1-3.6-4.4	10½	5	4.1	45	1340	1260	1420	1340	6.0
24	1-3.6-4.4	9½	1¼	4.2	45	1510	1730	1680	1640	7.9

The data of these tests are given in Table 5. The average strengths for the three different sand-aggregate ratios are shown in Fig. 2 (a), (b), and (c). For each sand-aggregate ratio the data furnish a satisfactory water-cement-strength relation. Furthermore, the relation is almost identical for the three cases. This is brought out by the fact that the same curve is fitted to the data in all three diagrams, (a), (b), and (c) of Fig. 2. For comparison the graph of the expression $\frac{14000}{7x}$ is shown in Fig. 2 (b). The strengths from the tests were about 50% greater than those given by this expression. Opposite each plotted strength a figure is given representing the average slump for these specimens. These tests made at Lehigh University check reasonably well with those

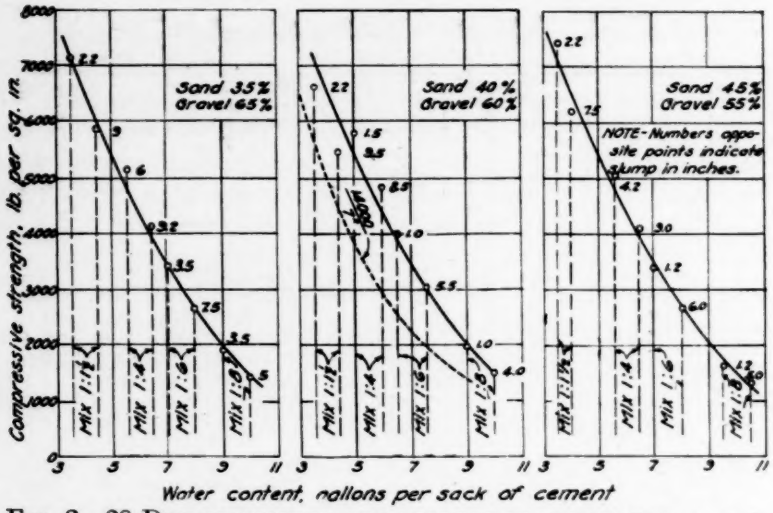
Sand 35% Gravel 65%
Sand 40% Gravel 60%
Sand 45% Gravel 55%
NOTE-Numbers opposite points indicate slump in inches.
Compressive strength, lb. per sq. in.
Water content, gallons per sack of cement
Mix 1:4½
Mix 1:6
Mix 1:7½
Mix 1:9

FIG. 2—28-DAY STRENGTHS OF CONCRETE PROPOSED FOR A. C. I. COLUMN INVESTIGATION

made by the Lehigh Portland Cement Co. and confirm the indications of uniformity found in their tests.

Since the majority of the columns are to be tested at an age of 56 days, the 28-day tests just described are not sufficient for determining all the mixes, and for this reason, another series of tests is being made. This series consists of four mixes of one water-cement ratio each, designed to give a sufficient range of strengths for the column investigation and to have consistencies close to those which are to be used in making the columns. The specimens will be tested at the ages of 7, 28, and 56 days. The data of the concrete and the 7 and 28-day strengths are given in Table 6. The strengths are also shown in Fig. 3.

TABLE 6—CONCRETE MIXES DESIGNED FOR COLUMN INVESTIGATION

Mix By Weight	Water Content Gal. Per Sack	Slump In.	Cement Content Sacks Per Cu. Yd.	Strength, Lb. Per Sq. In. At		
				7 Days	28 Days	56 Days
1 : 3.20 : 4.80	9½	4¼	4.2	760	1830	2100
1 : 2.30 : 3.45	7	6	5.7	1630	3810	4090
1 : 1.70 : 2.55	5½	6¾	7.3	2970	5390	5600
1 : 0.60 : 0.90	3½	5	14.8	5210	7280	7330

TABLE 7—PROGRAM OF TESTS FOR COLUMN INVESTIGATION

Series	Dia. in.	Concrete Strength Lb. Per Sq. In.			Reinforcement: Longitudinal Grade	Longitudinal Per Cent			Spiral Grade	Spiral Per Cent			End Condition of Columns				Storage		Method of Loading: Fast or Slow		Method of Loading: Long Time	No. of Columns: Like	No. of Columns: U. of I.	No. of Columns: L. U.	No. of Columns: Total
1	8		3500		Interm.			4	Interm.		1		Plain	Capit	P-20	P-30	Moist		Fast			4	8	8	16
2	8	2000	3500	5000	Interm. Struct. Rail	0		4	Interm.	0	1		d				Moist		Fast	Slow		4b	54	54	108
			3500		Interm.	0	1½	5	Interm.		1						Moist		Fast	Slow		4	12	12	24
			8000		Interm.	0	1½	4	Interm.	0	1						Moist		Fast			4b	11	11	22
3	8	2000	3500	5000	Interm.			4	Interm.	0	1	2					Moist	Dry			A. C. I.	4	72	72	144
		2000	3500	5000	Interm.		1½	5	Interm.			2						Dry			A. C. I.	4	24	24	48
			3500		Interm.		1½	5	Interm.	0	1							Dry			A. C. I.	4	8	8	16
			3500		Interm.		1½	5	Interm.			2						Dry			N. Y.	4	4	4	8
4	8	2000	3500	5000	Interm.			4	Interm.	0		2					Moist		Fast			3	0	18	18
		2000	3500	5000	Interm.			4	Interm.	0	1	2					Moist				.95, .9, .8, .7, .6	1	0	45	45
		2000	3500	5000	Interm.		1½	5	Interm.			2					Moist		Fast			3	0	18	18
		2000	3500	5000	Interm.		1½	5	Interm.			2					Moist				.95, .9, .8, .7, .6	1	0	30	30
5	12	2000	3500		Interm.		1½	4	Interm.		1						Moist		Fast			2	8	0	8
	20	2000	3500		Interm.		1½	4	Interm.		1						Moist		Fast			2	8	0	8
	28	2000	3500	e	Interm.		1½	4	Interm.		1						Moist		Fast			2	4	0	8
6	8a	2000	3500		Interm.			4	Interm.		1						Moist		Fast			2	4	0	4
	12a	2000	3500		Interm.			4	Interm.		1						Moist		Fast			2	4	0	4
	20a	2000	3500		Interm.			4	Interm.		1						Moist		Fast			2	4	0	4
	28a	2000	3500	e	Interm.			4	Interm.		1						Moist		Fast			2	2	0	4
7	8		3500		Interm.	0	1½	3	High Y. P.		1						Moist			Slow		4	6	6	12
			3500		Interm.	0	1½	4	High Y. P.			2					Moist			Slow		4	6	6	12
			3500		Interm.	0	1½	1	Interm.			2					Moist			Slow		4	6	6	12
																					Totals—		245	322	573

General Note: In all cases the length of column is 7.5 times the diameter.

a Diameter of spiral. 2-inch shell outside spiral.

b Plain columns are in triplicate at each laboratory.

c One year under working stress specified by A. C. I. and N. Y. respectively.

d End conditions of columns for series 2 to 7 to be determined after series 1 are completed.

e Make at Bureau of Standards, or omit.

To assist in a comparison of the two series of tests the curve fitted to the results shown in Fig. 2 is repeated in Fig. 3, also the graph of the equation $S = \frac{14000}{7^x}$.

The program of column tests as laid out is given in Table 7. An effort has been made to make the table self-explanatory. It

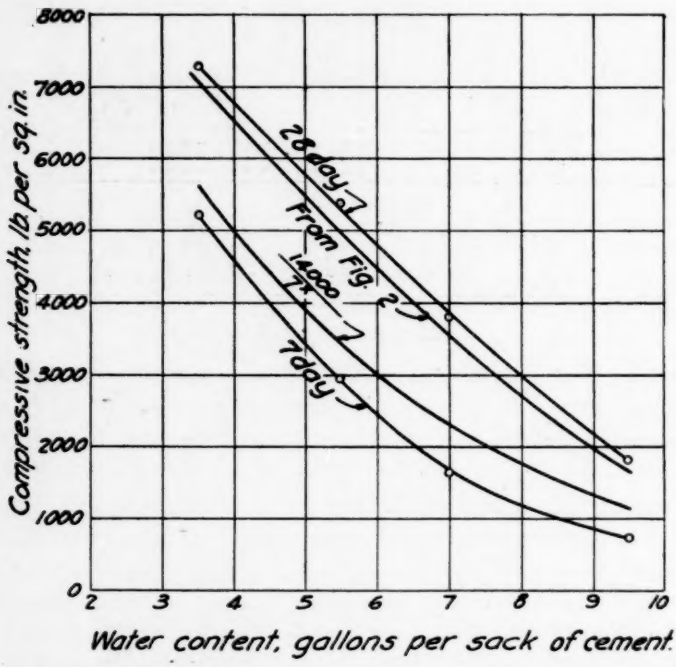

FIG. 3—7-DAY AND 28-DAY STRENGTHS OF CONCRETE DESIGNED FOR COLUMN INVESTIGATION

will be seen that most of the 8-in. columns are to be made in duplicate series at the University of Illinois and at Lehigh University. With the plain columns three, and for most other cases two, companion specimens are to be made at each of these laboratories. Probably more companion specimens would be desirable, but the expense involved seems to make this impracticable.

According to the present schedule the columns will be made in the following order: series 1, 3, 2, 7, 4, 5, and 6.

CONVENTION DISCUSSION

Chairman Lindau: This report now is open for discussion. It might be well for those who have not followed this matter as closely as they would like, if Professor Slater or other members of the committee would give a general idea of what is hoped to be accomplished by these tests. When we started out to raise funds for the tests, the explanation was made that this is not just another series of column tests. I think that might be enlarged upon for the benefit of those who are here and also for the benefit of any discussion that may follow.

Professor Slater: I do not know that I can enlarge upon it in a way that will be convincing, but I will make an effort. In the first place, I mentioned in giving the report, that the total number of columns tested to date in this country and Canada, as far as we have been able to find the results, is 572. Our program (there was no effort at matching the number or beating the number at all) as laid out calls for 573, one more column than all those that have been tested in this country up to date as far as we know. The committee does not consider the merit of the program merely in the number of columns tested; we believe that by having all of these tested in one program, we do not have to repeat so many of the control specimens as we would if we tested a group of fifty in one series of tests and another fifty in another series of tests, etc. The value of the results should therefore be far more conclusive than that of the same number of tests scattered through the series shown in Table 1. The columns in general are eight inches in diameter and five feet long. I expect that will seem a rather small column on which to base a great deal of important information. However, we are supplementing the investigation with some larger columns, to as high as a diameter of 28 in. That is a spiral diameter, of 28 in. with a shell of two inches outside of that. The lengths of all the columns are seven and a half times the diameter, giving five feet for the 8-in. columns and corresponding lengths for the larger columns.

The first series of tests is aimed at finding out something of how we are going to have to test the columns in order to secure the proper application of load in the column. There are to be columns with enlarged capitals to avoid the possibility of end

failures and columns with no enlargement of the end. Then again there are to be columns with dowels in the ends to distribute the load into the shaft of the column. It is expected that as a result of the preliminary series of tests, we will be able to decide definitely on the best method of making the remainder of the columns. At the University of Illinois the columns for the preliminary tests have already been made and at Lehigh University are waiting merely on materials, so it is expected that this preliminary series of tests should be out of the way in the course of the next two months or so.

One of the incentives, perhaps the most striking incentive, in going into such an investigation as this is furnished by the results that have been found when measurements are taken on the reinforcement of a column that has been under load for a long time. You know that strains have been reported which correspond to the stresses in the reinforcement as high as twenty-five, thirty, thirty-five and I believe even forty thousand pounds per square inch in the columns under working load. The fact that in certain instances the concrete had been partially unloaded upon the reinforcement is brought out in this way in a striking manner and confirmed by other data. There is a report in Morsohs' book, "Der Eisenbetonbau," in which, in Germany, a number of years ago, in the effort to determine whether or not columns of a building were strong enough to support an added story, they investigated this matter by removing one of the columns and subjecting it to a test. On the removal of this column they found that the reinforcement elongated and cracked the concrete entirely across the column at intervals of about two feet to the best of my recollection, along the length of the column, showing that the steel undoubtedly was taking a high stress and that the concrete had yielded. We call it flow. Some object to the term "flow," but whatever it is, there had been a yielding of the concrete in unloading the stress upon the steel. Column formulas based on the assumption of a constant ratio between the modulus of elasticity of the concrete and that of the steel, generally seem to penalize the stress in the steel in order to avoid over stressing the concrete with the ratio of the modulus of elasticity ordinarily assumed. With the phenomena I have just mentioned, stressing the steel to forty thousand pounds a square

inch, which indicates to us that we need have no worry that the steel is not working to as high a stress as we want it to work. It does introduce some interesting problems as to how much the strength of that column is affected and this investigation includes some long time tests in which the column will be kept under load for a year, some of them under moist conditions and some of them under dry conditions, with strains being measured at intervals throughout this length of time, and at the end of the time the strength of the column will be determined. Now there are a great many other features of the program that I could take considerable time to discuss. I am afraid that if I kept on now, I might wander along without hitting on those which are most interesting.

Readers are referred to the JOURNAL *for October, 1930 (Vol. 27, No. 2) for discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1930.*

The Coloration of Concrete

*Report of Committee 703**

BY RAYMOND WILSON†, AUTHOR-CHAIRMAN

This report was written by Mr. Wilson, Author-Chairman Committee 703, Color in Concrete Products. Maximilian Toch and O. A. Malone, critic members of the committee, contribute discussion. The subject matter of the committee's work is open to general discussion. Such discussion should be available by Sept. 1.

THE INCREASING importance of color in daily life has made the use of colored concrete an architectural necessity. It is the purpose of this paper to present a review of the principal means of achieving chromatic effects and to point out the precautions which experience and tests indicate as necessary in the use of each method.

The methods of coloring concrete may be considered in three classifications: first, the integral admixture of finely divided, inert color pigments; second, the use of colored aggregates which are exposed by some treatment of the surface after the mixture has hardened; third, surface treatments which may be further subdivided into penetration treatments and cover coats such as paints. The pigment admixture method is probably the most extensively used of the various methods listed. It is used in stucco, many pre-cast products, sidewalks, driveways and floors. The colored aggregate method has been used with conspicuous success in monumental structures and in cast stone, lighting standards, art marble and terrazzo, frequently in combination with pigments. The penetration processes have been applied to floors and tennis courts having a troweled surface. The use of paint coats on concrete is not greatly developed, though much

*Presented at 26th Annual Convention, Feb., 1930.

†Associate Chemist, Research Laboratory, Portland Cement Association, Chicago.

has recently been accomplished in interior decoration by that means.

COLORING WITH PIGMENTS

Not all pigments which can successfully be used in paints are suitable for cement colors. Broadly speaking, the requirements for cement color pigments are color durability under exposure to sunlight and weather, fine subdivision, intensity of color when finely subdivided and a chemical composition such that the pigment does not interact with the cement to the detriment of either cement or color.

These requirements are generally best met by the following types of pigments:

Iron oxides: Red, black, yellow, brown
Manganese dioxide: Black
Chromium oxide: Green
Carbon pigments: Black
Ultramarine blue: Blue

Organic colors have not been developed for use with cement to a point where they can be considered dependable. Other colors which should be avoided are those containing Prussian blue, cadmium lithopone and zinc and lead chromates.

By the use of color pigments it is possible to obtain color effects over a fairly wide range of hues. Brilliant or intense colors are not generally attainable. Black is difficult to produce where the surface is floated or trowelled as a final treatment. Some strikingly good blacks have been produced in concrete with special surface finishes. The carbon black pigments, which are most effective weight for weight, cause serious reductions in strength and are difficult to incorporate uniformly in the mix when the quantity used exceeds two or three per cent of the weight of the cement. Black iron oxide is probably the best pigment for producing black concrete, but the greater quantities necessary make the cost rather high.

There are great differences in the tinting qualities of pigments which makes the question of relative economy of high priced and low priced pigments of importance to the user. A quantitative rating of tinting qualities would probably, in the author's belief,

show an advantage for the better quality pigments, even though the price per pound is higher.

The quantity of pigments which may safely be used is customarily limited to about ten per cent of the weight of the cement (two per cent for carbon black). Data have been obtained which show that these limits may be considerably exceeded without injury to strength, at least under certain conditions and with certain pigments. The composition of the mixture, particularly with regard to the amount of fine material in it, will doubtless influence the effect of pigment admixture on the strength. Excessive amounts of pigment would probably impair the durability of concrete, though the author has no conclusive evidence on this point. On the other hand, in a series of exposure tests of pigment-colored mortars at the Portland Cement Association Research Laboratory the colored mortars withstood four years weathering as well as the uncolored mortars.

An essential in the use of color pigments is that they shall be thoroughly incorporated in the mixture. Most pigments have a tendency to form soft aggregations which can be broken up only by very thorough mixing. While it is true that addition of the color pigment to the concrete at the time it is mixed can result in good distribution, there is the strong possibility that streaked and spotty color effects may be produced.

Unquestionably the best method of insuring thorough incorporation consists of premixing the cement and the color. Another reason for using premixed cement-color mixtures is found in the fact that by far the most important single factor affecting the color produced by combining a given pigment with cement is the ratio of pigment to cement. For well organized shop work, color and cement can be carefully proportioned, and mixing accomplished in suitable units, such as pebble mills. For the field operation colored cement is now available in a number of standard colors which can be mixed or diluted with uncolored cement to provide variations. Colored cement is made by grinding portland cement with pigment, which if properly done, insures thorough incorporation and uniformity not ordinarily obtained on the job, particularly the smaller job.

The choice of materials other than pigment depends upon the requirements of the work. For the darker or more intense colors

gray cement is generally used. White cement or a mixture of white and gray cement is necessary if the more delicate tints are sought. For these latter types the use of aggregates such as silica sand and marble is advisable, since fine material in the aggregates, if of a hue different from that of the pigment used, is likely to affect the color of the mixture. The usual result is that instead of the clear color obtained with clean materials a muddy appearance is produced.

Aggregates of color harmonizing with ground mass of colored cement paste are, of course, an essential if the finish is one which exposes the aggregate to view or if the surface film of cement is likely to be removed through the agencies of weathering.

Lack of permanence of color in a concrete surface is, in the author's opinion, less often due to deficiencies in the quality of the pigment than to other causes. Exposure of aggregate particles may result in apparent fading of the color, even though the color of the cement paste is practically unchanged. At a distance too great to distinguish individual aggregate particles, the apparent color of the mass is the resultant effect of the colors of the exposed aggregate and of the cement paste. If exposure is well advanced, the aggregate forms the greater part of the visible surface and its color will dominate. The best assurance against a possible marring of the color effect from this cause is the use of aggregate which, if exposed, will not result in an unharmonious appearance. The aggregate may be one of the same color as the pigmented cement paste, or it may be a colorless or white aggregate.

Efflorescence is another cause of color impermanence. A light deposit may cause the color of the concrete to appear faded or the color may be completely masked by a heavy one. Obviously the effect of efflorescence is more strongly apparent on darker or more intensely colored concrete than on a delicately tinted surface. The sources of the efflorescence are known and some of the factors affecting its occurrence are understood but methods for its certain prevention have not been completely developed. In its most common form efflorescence consists of calcium carbonate, though occasional deposits of sodium and potassium carbonates and the sulfates of sodium, potassium,

calcium and magnesium have been found. The water in the pores of concrete contains some soluble material extracted from the constituents of the concrete. The water always contains calcium hydroxide, and generally the hydroxides of the alkali metals. Evaporation of the water causes the dissolved material to travel to the surface at which evaporation takes place, where it reacts with atmospheric carbon dioxide to form carbonates. The calcium carbonate deposit formed from calcium hydroxide is only slightly soluble and is characterized by close adherence to the surface, whereas the deposits of alkali carbonates are readily soluble and generally fluffy and loosely attached to the concrete.

It is seen from this brief description of the mechanism of efflorescence formation that an essential to its development is the passage of water from the interior to the surface of the concrete. The most promising means of eliminating efflorescence are those which tend to minimize the passage of water. Proper methods of construction and concrete of uniform high quality are of course necessary and possibly water-repellent integral waterproofers or surface seal coats will be required.

COLORED AGGREGATE

The production of color effects in concrete through the agency of colored aggregates requires a treatment which exposes the aggregates to view. This may be accomplished by cutting or grinding the concrete to produce a smooth surface or by an acid wash which removes the cement, leaving a surface the texture of which is determined by the size, amount, and distribution of the aggregate.

Exposure of the aggregate by cutting or grinding is exemplified by terrazzo, cut cast stone and the like. In this type of product, pigment coloration also is frequently used, the aggregate being chosen to provide irregular areas of contrasting color or of a different shade from that of the cement matrix. Grading of the aggregate is an important factor in the effects produced. A common expedient for emphasizing the color of the aggregate is the use of relatively coarse aggregate with a fairly fine sand, leaving a gap in the grading. Color effects are limited only by the colors of the aggregates and the ingenuity in combining them.

Where the acid treatment for exposure of aggregates is used, the same methods of obtaining definition of the aggregates are used. Certain types of precast work are composed of a facing made up of cement and an aggregate of uniform size, and a backing of concrete or mortar. This results in the production of a visible surface made up almost entirely of aggregate. By careful choice of aggregates uniformity and permanence of color can be achieved. A notable advantage of this method of coloration is that the concrete is almost free from the marring effect of efflorescence because of the large proportion of aggregate in the visible surface.

PENETRATION PROCESSES

Coloring of concrete by penetration processes has been an attractive possibility but the difficulties in the way of its successful application have limited its development. It has been rather extensively used on hard trowelled surfaces such as floors and tennis courts. The process depends on a reaction between a constituent of the penetrant and a constituent of the concrete with the formation of a colored product in the surface pores. As an example, treatment of concrete with a ferric iron salt results in the precipitation of hydrous ferric oxide which is reddish brown in color. A fairly wide range of reds, yellows, browns, greens and grays can be produced by the salts of various metals. The variations inherent in the absorptiveness of the surface cause some slight non-uniformity in the color.

In practice it is customary to incorporate a fluosilicate hardener with the color-producing salt. The solution is applied to the concrete after the latter has been cured and allowed to become surface-dry. Several applications are usually made allowing time for penetration and drying between treatments. After the final application, an oil-wax treatment is applied to fix and protect the color.

Good results with this type of coloration depend on the quality of the concrete, as well as upon the maintenance of proper composition of the solution, and skill in its application. For that reason it is customary for the company which colors the floor to take the contract both for placing the wearing course and coloring it.

The information relative to this process has been furnished for this report by Dr. L. B. Miller, Director of Research of the Master Builders Co., Cleveland, Ohio.

PAINTS

Painting on concrete has not been uniformly successful. The elements for satisfactory performance of paints on concrete are not fully known, and comparatively little published data are available in this field.

A consideration of the chief points of difference between a concrete surface and the surfaces of other materials on which paint is more extensively used suggests as possibly important factors the absorptiveness and the chemical composition of the concrete.

Concrete when dry enough for painting is sufficiently absorptive to remove from a paint a rather large part of its liquid vehicle. If a paint of the usual proportions of pigment and vehicle is applied to concrete, the resultant film is deficient in vehicle and would therefore have a low degree of durability.

In addition to its absorptiveness concrete contains calcium hydroxide which, under certain conditions, may saponify the oil which forms the major part of the vehicle. Upon exposure to air the hydroxide is converted to calcium carbonate which is not likely to affect the paint vehicle. Neutralization of the hydroxide in the surface layer may also be effected by other means; the best known method consists of treatment with zinc sulfate solution.

The minimum requirements in the preparation of concrete surfaces for painting have not been established. The present recommendations of the Portland Cement Association are that:

1. The concrete, if new, be treated with zinc sulfate solution.
2. The surface be dry.
3. The final coat be preceded by two filler or priming coats the first of linseed oil, and the second of paint well thinned with linseed oil.

A third factor which would militate against the successful use of paint is the possibility of passage of water through the concrete toward the painted surface. Interior or partition walls would not ordinarily be affected but interior faces of walls exposed to pre-

cipitation would need to be free from areas subject to percolation to permit satisfactory performance of paint.

The durability of paint on concrete exposed to the weather has received but little attention. If so applied that a film of good quality results, a high grade paint would appear to have as good a chance for survival on concrete as on any other surface. This conclusion is verified by successful performance of paints applied in accordance with the methods suggested above.

It is difficult to make any general statement covering the question of paints of compositions markedly different from the usual oil-pigment types. Some manufacturers have promoted oil paints with portland cement as the main constituent of the pigment and numerous water paints are on the market. When portland cement is an important constituent of a water paint the difficulties of affording proper curing to a thin cement film are encountered. The testing of water paints has received very little attention and the record of experience is almost the only basis from which the interested potential user might draw a conclusion.

SUMMARY

The principal known methods of coloring concrete have been briefly discussed in this report. The choice of the basic method to be used is dependent on the requirements of color and finish, the type of service and exposure and the relative costs. No attempt has been made to present detailed methods of obtaining definite color effects. In the final analysis, cook-book technic is not a sufficient equipment for successful use of color in concrete. Detailed application of the few known principles to a specific work will disclose lines of improvement to the alert observer.

BY MAXIMILIAN TOCH*

A number of the pigments in use, were described by the writer in an address delivered at the Institute convention in February, 1925.†

Two things must be borne in mind in the use of pigments for cement. The one is that no pigment should be used which will

*Toch Bros., New York City.

†"Shall Anything Be Added to Portland Cement," Maximilian Toch, *Proceedings*, A. C. I., Vol. 21, p. 134.

basically combine with the free lime as it is generated, and secondly, no pigment must contain soluble salts which will eventually mask the color.

In the first instance, all of the copper salts and all of the lead salts are dangerous in cement work. The cadmium colors are not dangerous if used in limited quantities, and are exceedingly brilliant and permanent to light. Any color that contains calcium sulphate lowers the strength of the concrete to a remarkable degree, and as this material is soluble in water one to 700, it produces efflorescence in almost every case.

The writer agrees with Mr. Wilson that the pigments should be mixed with cement by fine grinding. The best results are obtained that way.

As regards those pigments that produce the best results chemically, there are many among them that are not so good physically, as for instance, any pigment that has a higher specific gravity than portland cement is very likely to sink in the mixture instead of float on the surface. This is overcome by one manufacturer who supplies the physical principle of flotation by mixing the pigment in a small percentage of gelatinous alumina base.

There are several organic pigments which give excellent results, particularly the Para, Chlor Para and Toluidine reds.

Efflorescence on the outside of a building is not so easy to overcome if it is produced by the solvent action of rain water from the back of a parapet wall; no coating on the front will overcome it. If it is produced through the crystallization of an excess of soluble salts on the face only and not through filtration, then one of the stearate solutions containing over 12% of solids, frequently is a good remedy, but in a few cases it also fails.

The use of raw linseed oil on a concrete structure is ill-advised, owing to the fact that the fatty acid and the lime forms soluble mushy salts, which on repeated washing or submersion in water, is very likely to soften the concrete.

Painting concrete has made very great strides in the last fifteen years, but not through the use of linseed oil paints. The well-known reaction of the China wood oil acid copal resins is the proper base, as a calcium resinate is formed which gives a hard and flint-like surface. Thousands of gallons have been used on large structures like the Panama canal, Muscle Shoals and other

power houses, with very lasting results, but in no instance has a slow-drying paint based on linseed oil, been found effective. A floor treated with a thin solution of acid resin China wood oil is the best method for wear-proofing and bringing out the color uniformly.

Sulphate of zinc as a neutralizer is not as good as the fluosilicate. The fluosilicate of zinc and magnesia appears to be the best, but even at that, the penetration of both of these depends upon the density of the concrete.

Mr. Wilson deserves a great deal of credit for the article which he has written, which is an excellent introduction to the entire subject.

BY O. A. MALONE*

If the report is meant to convey the idea that we may now have it as an "every day diet," in my opinion, it is wrong. On the other hand, if it means that under proper, intelligent supervision these things may be done with certainty, in my opinion, the paper is right. In other words, it depends almost entirely upon the knowledge of those in charge of the work. I do not believe that we are quite ready to classify coloring mass concrete a commercial success. It is still, more or less, the work for a specialist.

Readers are referred to the JOURNAL *for October 1930 (Vol. 27, No. 2) for discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1930.*

*California Stucco Products Co., Los Angeles.

Shore and Storm Protection on the Gulf Coast*

BY J. B. CONVERSE†

It has been my privilege recently to read that very excellent paper of Commander G. T. Rude of the U. S. Coast and Geodetic Survey on "Tides and their Engineering Aspects," for which he was awarded the Norman Medal by the American Society of Civil Engineers. In the discussion that followed this paper I was impressed most forcefully by a statement of William Barclay Parsons to the effect that "The most important factor for the engineer thoroughly to understand is that he cannot generalize with safety in respect to what will take place in tidal action in any one locality." How well this applies to shore and storm protection on the Gulf Coast is realized when we consider Galveston, Texas, subjected to wave action from a storm that may travel one thousand miles across the Gulf unobstructed, when we consider the Mississippi Coast protected by a chain of islands ten to twelve miles off shore and then such places as Port Arthur, Texas, sixteen miles inland on Sabine Lake but subjected to storm tides that would submerge it with water from two to eight feet deep. In this paper no attempt is made to analyze tides, ocean currents, wave action, the action of sea water on concrete or the many other factors entering into all coastal works, but it merely reviews conditions as they exist and have been met on the Gulf Coast.

You gentlemen are familiar with the appalling loss of life and property that has occurred in these tropical hurricanes on the Gulf Coast in recent years, however, I wish to recall that in the 1900 Galveston storm 6,000 lives were lost and property damage amounted to $30,000,000.00.

In approaching the question of shore and storm protection on the Gulf Coast, it is well first to review the action of tropical hurricanes in the Gulf of Mexico. These hurricanes enter the

*Presented at the 26th Annual Convention, February, 1930.
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FIG. 1—GALVESTON, TEXAS, GRAVITY TYPE WALL, CONSTRUCTED SOON AFTER THE STORM OF 1900

Gulf through the Yucatan Channel or the Florida Straits, and they move toward the coast at the rate of about ten to fifteen miles per hour and with a counter clockwise rotary motion. The winds in these hurricanes attain velocities from 90 to 120 miles per hour and in rare instances velocities of 150 miles have been recorded for a few minutes. The storm tides that accompany these hurricane winds are the greatest destructive agents to the shore line of the Gulf Coast. The winds in these hurricanes are of much greater intensity in the two quadrants to the right of the path of the center of the storm. Currents are produced in the waters of the Gulf moving from the right to the left of the path of the center. As the hurricane passes inland over the coast line, the waters to the right of the center are driven with great force upon the shores. Their return to the open waters of the Gulf in the counter clockwise rotary direction of the hurricane is obstructed by the elevation of the shore line. The result of this is that storm tides to the right of the center have been known to reach an elevation of 18.0 above M. G. L., their height depending upon the intensity and duration of the storm and the topography of the coast effected. The effect of this height of 18 ft. can be

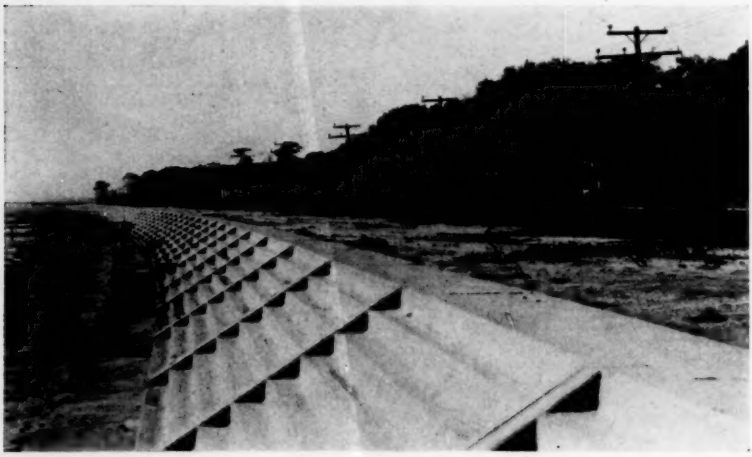

FIG. 2—HARRISON COUNTY, MISSISSIPPI, SLAB TYPE WALL, SUPPORTED BY SHEET PILE CUT-OFF AND SQUARE CONCRETE PILES; 1925—DESIGNED BY H. D. SHAW, GULFPORT; A. M. SHAW, NEW ORLEANS, CONSULTANT

appreciated when you consider that the normal daily variation between high and low tide on the Gulf Coast does not exceed 2 feet. To the left of the center the winds are off shore and result in a tide below M. G. L.

A few specific instances of the banking up of water along the coast are of interest. The City of Mobile is on Mobile Bay, thirty miles from its entrance to the Gulf at Fort Morgan. The ground elevation at the entrance of the bay approximates 2.0 and at Mobile 15.0. The 1916 storm passed inland so as to place both Mobile and Fort Morgan just to the right of the center. The maximum storm tide elevation at Fort Morgan on the Gulf was 5.3 while at Mobile an elevation of 11.6 was reached or a difference of 6.3 feet. At Galveston in 1900 the hurricane passed inland just to the west of Galveston and produced a storm tide of 15.0. The 1915 storm passed inland with Lake Ponchartrain just to the right of the center. The storm tide elevation at the Rigolets and Chef entrances to the lake was 9.5 while at the western end of the lake an elevation of 13.0 was reached. The storm of 1909 entered the low marsh lands of South Louisiana

just east of Morgan City; this storm to the right of the center reached an elevation of 10.0 while 75 miles to the left of center the elevation was 0.1.

The first question to be determined in the design of any shore protection works is what storm tides may be expected. The records of the U. S. Weather Bureau on tropical hurricanes are very extensive. A study of these together with local data will determine whether the locality under consideration has ever been in a position to receive the maximum storm tides of any hurricane in that vicinity. If it has been, a comparison can be made of the intensity and duration of that storm with storms of maximum recorded intensity and duration in the Gulf, and a decision made as to possible storm tides. If the locality has never been in the path of maximum storm tides from a hurricane, an estimate of storm tide elevation should be made assuming that a storm will pass inland just to the left of it: taking into consideration the topography of the adjacent land, storm tides from storms that have passed inland within say one hundred miles to the left of it, and also if the project lies on a tidal estuary or bay, consideration of the direction of the inlet with respect to the paths of hurricanes in that section of the Gulf. Having arrived at a decision as to the storm tide elevation and knowing the general elevation of the ground along the shore to be protected we can now classify the particular shore protection under one of three distinct divisions.

1. A shore line with adjacent ground surface elevation so low that hurricane storm tides will submerge it.

2. A shore line with adjacent ground surface elevation so low that storm tides would submerge it but which for the protection of life and improvements must be kept free of hurricane storm waters.

3. A shore line with adjacent ground surface at an elevation above storm tides.

Before entering into a discussion of these three divisions your attention should be called to the fact that shore protection on much of the Gulf Coast means a protection of such a nature as to permit building and maintaining a paved highway along the shore.

Division No. 1 comprises those large undeveloped areas along the Gulf Coast that lie from elevation 2.0 to elevation 4.0. To

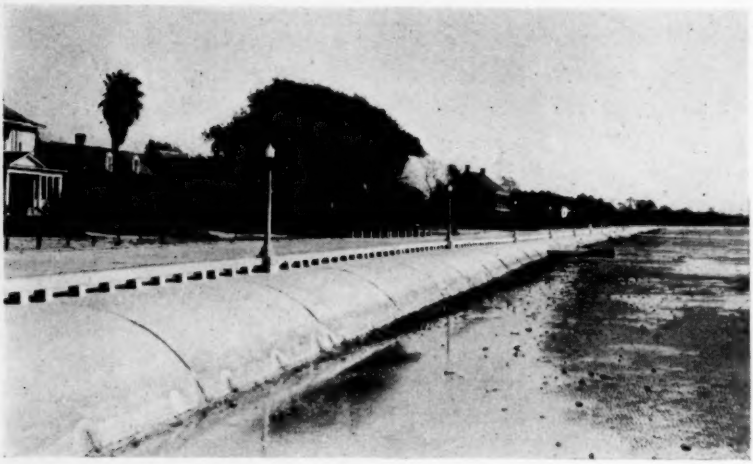

FIG. 3—JACKSON COUNTY, MISSISSIPPI, 1926—DESIGNED BY F. MCGOWAN. CURVED SLAB WITH SHEET PILE CUT-OFF WALL AND NO REAR SUPPORT

attempt to prevent these areas from being submerged under storm tides of eleven feet or more would entail expenditures that at this present stage of development would be prohibitive. It is desirable at this time, however, to continue coast line highways along this shore uninterruptedly. The elevation of the shore protection (truly in this case road protection) is determined by the elevation of the road surface. In as much as it is desirable to have this highway open for travel at all times except during storms a grade is usually established based on maximum tide exclusive of hurricane tides. From the view point of storm damage this grade should be as low as is consistent with maximum travel under average conditions as in this manner less obstruction will be offered to wave action and at the height of the storm the highway and its protection works will be submerged to such extent as to be undisturbed by wave action. Your attention is called to the manner in which a storm tide covers the low marsh lands and recedes with no appreciable effect on this area. In such cases as this an impervious surface such as a concrete slab between two rows of sheet pile wall is one solution of the problem. The elevation of

the top of the sheet piles corresponds to the elevation of the highway, the space between rows being filled and forming a subgrade for the slab. Special care should be taken to seal the joint between slab and pile. It is needless to say that any structure subjected to wave action and which is dependent in any respect for stability on an earth fill must be constructed so as to prevent these waters from penetrating.

Division No. 2 embraces the populated areas of the Gulf Coast which in some cases would be submerged with six to eight feet of water and shore protection becomes storm protection. In this case in addition to protection from hurricane tides provision has to be made to drain the protected area in case of heavy rainfall which usually accompanies a hurricane. In some localities drainage under normal conditions is by gravity through the protection wall. With the rise of the storm tide the flood gates are closed and pumping is necessary. The height of the protection work in this case is determined by the elevation of maximum storm tide, and the extent to which the structure is subjected to wave action. The type of structure may vary from a gravity type concrete wall to the reinforced concrete slab type.

Division No. 3 includes those shores where the elevation of the ground surface is above hurricane storm tide and affords an ideal situation to combine the shore protection works with a coastal drive. This combination eliminates a serious trouble that has been experienced with some coastal protection work in the past. When a wave strikes a wall it is deflected upward; during a hurricane the winds carry this water in shore and it is deposited behind the wall with a destructive force. An impervious slab should be provided behind the wall to prevent the undermining action of these falling waters. This slab then serves a dual purpose in that it also provides an excellent highway surface. The type of structure here may also vary from a gravity type concrete wall to the reinforced concrete slab type.

For any particular locality the height of the wall structure having been determined by conditions previously outlined, the next question to be considered is the depth of erosion to be expected at the face of the wall. This requires a study of soil conditions varying from the sand beaches of the eastern Gulf to

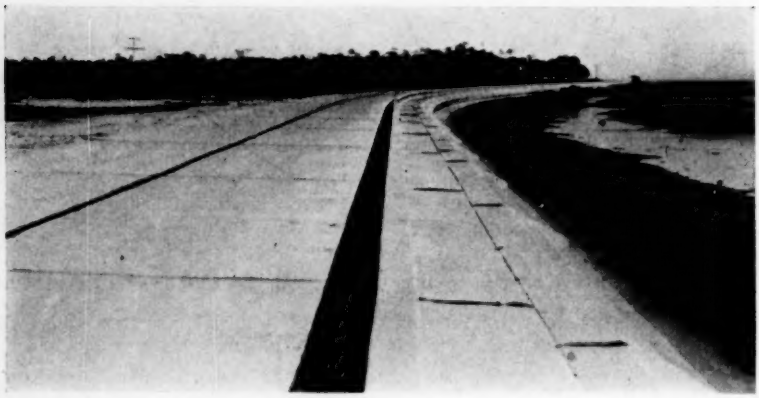

FIG. 4—HANCOCK COUNTY, MISSISSIPPI—1926; DESIGNED UNDER SUPERVISION OF J. B. CONVERSE, AS ASSOCIATE OF J. W. BILLINGSLEY, NEW ORLEANS. A SLAB TYPE WALL SUPPORTED BY SHEET PILE CUT-OFF WALL AND SQUARE CONCRETE PILES

the clays of Texas estuaries. Consideration should be given to the possible height of waves as their effect is seldom felt at a depth greater than their height. The proximity of dredged channels, tidal currents, littoral drift, and wave wash from winds and vessels, are all factors.

With the height of structure and depth of erosion determined the character of the back fill known and an assumption made as to the dynamic pressure from the waves, the design of the wall follows the theories of well recognized engineering practice.

In considering types of concrete walls for shore protection there are the unreinforced gravity type and the reinforced slab type. The gravity type with no steel eliminates, of course, any question as to failure due to corrosion of reinforcement. In my opinion present methods of design and control of concrete mixtures together with a net covering of two to three inches for reinforcement plus rigid field inspection will prevent corrosion of reinforcement in the reinforced slab type. I have recently had an opportunity to inspect a reinforced concrete sheet pile wall on the Gulf Coast that was constructed in 1912. The reinforcement has a net covering of 1½ inches. This wall now 18 years old is

in an excellent condition. Most of the shore protection work on the Gulf Coast consists of reinforced concrete due to the economy of this design.

There are a few points that may well be mentioned in considering the reinforced concrete slab type. This type as constructed on the Gulf Coast consists first of a concrete sheet pile cut off wall. This cut off wall is carried as a rule to a point above average daily high tide. It is essential that the joints between piles be sealed so that there can be no seeping out of the back fill. This has been accomplished by driving tongue and grooved sheet piling. The tongue does not extend the full length of the pile, both sides of the piling being made with the groove in the upper section. After driving, this cylindrical space made by the grooves, is cleaned out and grouted. From the cap of this cut off wall a slab is carried at an inclination slightly less than the angle of repose of the material composing the back fill for the wall to a beam running parallel to the wall. This beam is in turn supported by concrete piles. On the greater portion of the Gulf Coast construction these slabs have been constructed with steps running the full width of the slab which makes the beach in front of the wall easily accessible at any point. A saving in construction can be effected and an additional height of protection can be secured by making the sidewalk adjacent to the wall flush with the top of the wall. The roadway slab can then be elevated 10″ to 12″ above the sidewalk by means of a curb and guard rail, and carried on a slope to its inner edge. In this manner spray falling on this slab returns directly to the Gulf over its surface and the face of the wall which eliminates the installation of all catch basins and drainage lines that would be necessary were this roadway depressed in the usual manner of street construction. Such matters as bending in the cut off piles should be carefully considered if erosion will take place, as well as excessive pressure due to hydraulic fills. From the practical side of constructing these walls we sometimes meet a condition that is unfortunate in that there are some certain stretches on the Gulf Coast which within a short distance will have a shore line with an out cropping of sand clay on a beach that is in general sand. These cut off piles on Gulf Coast work have been placed with a jet and we find that when the sand flows back and is compacted around them

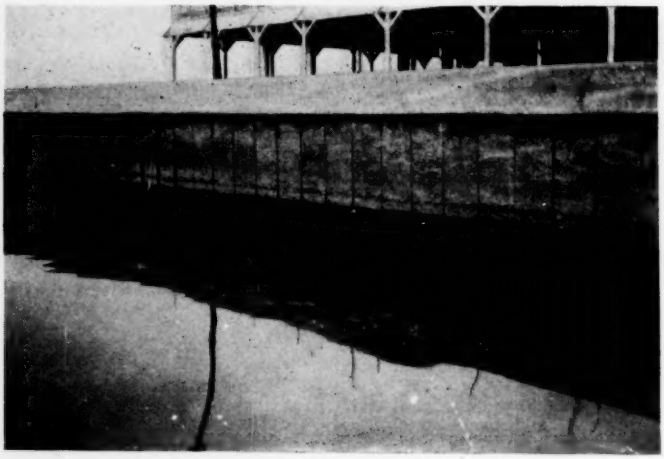

FIG. 5—CONCRETE SHEET PILE WALL, WITH TIE BACKS, PORT ARTHUR, TEXAS. CONSTRUCTED 1912.

there is no movement whatsoever. However, when the clay enters into the situation and the contractor is equipped for jetting only, the pile is left in a cavity that is not filled in the same manner as those that are jetted in the sand. In my opinion, in those stretches where clay does exist, the use of the jet should be prohibited.

On a project that I am now handling, the production of a concrete with a high density, which would protect the reinforcing steel from the action of sea water, was a prime factor in the design. This was sought through the means of a high cement ratio; the limiting of the water content to five and one-half gallons, and the careful supervision of the mixing and placing which is necessary on work of this nature.

The Joint Committee recommends a seven bag mix for concrete exposed to the action of sea-water. It was desirable to use a 1:1½:3 mix with 5½ gallons of water with a minimum of 7 bags of cement to the cubic yard of concrete. Considering the fact that the quality of the concrete was of the utmost importance, my desire to make the specifications as specific and rigid as practical considerations would allow is readily understood.

To avoid writing a specification calling for an impossible mix, a mix fulfilling the desired requirements was attempted. The mix

proved not only practical, but desirable in every way. The concrete was workable to such a degree as to offer no unusual difficulty in working around the reinforcing steel; the seven day strength averaged 3,000 pounds per square inch; the twenty-eight day strength 4600 pounds per square inch; and the absorption at 28 days was 3.48% by dry weight. That these tests were very favorable is evident when you consider that a working stress of 800 pounds per square inch was used.

The desirability of a large factor of safety was also kept in mind in the design of the wall proper. The stresses were kept as low as was economically possible. In dealing with earth pressures, conditions were assumed that it is reasonably certain are much worse than those that will exist after construction is completed. The piles were designed for extreme handling conditions with a 50% increase in stresses to keep to a minimum the cracks that might occur in handling and go undetected notwithstanding the most rigorous inspection.

I wish to say here, that the great service rendered by the research laboratories in their efforts to improve the quality of concrete and extend its use, has in my opinion been of inestimable value to the practicing engineer in the design and building of successful shore protection works.

Very few engineers, if any, have at their command either the means or the facilities for such experimental work, and if they had; one could easily imagine the chaotic mass of contradictory information and opinion that would result. In contrast to the above unhappy condition, these laboratories, with the most modern testing apparatus, and under the direction of the best talent obtainable, have worked continuously at the problem, and have published their findings so intelligibly that it would be difficult for an experienced engineer to make a serious mistake as long as he follows the fundamental methods and principles that they advocate.

On the Gulf Coast proper very little work has been done in maintaining beaches by the installation of groynes. My experience has been very limited in this construction, however, I have installed this type of protection for causeways in our southern waters and have succeeded in building up the shore very materially. There is no doubt that where a littoral drift exists or

a prevailing current can be checked on a sand beach that this construction will prove most beneficial in maintaining the shore and in causing accretions.

For many years the Gulf Coast made various attempts to finance the building of structures to protect the properties and highways along the beach front. No works of any great magnitude were constructed for a long time due to the fact that the properties adjacent to the beach were not of sufficient value to support the bond issue required. In addition to this there was the question of the equity in assessing these costs entirely against these properties. It was not until legislation was passed under the head of "Road Protection," and a refund to the coast counties from the gasoline tax was permitted, to finance the construction, that any large projects were started in the eastern Gulf sections. This gasoline tax under a road protection law has probably provided more funds than any other form of financing. Another method of financing by which large projects have been installed in Texas is through legislative acts under which certain populated sections of the coast were granted a remission of a portion of the State Ad Valorem Tax to provide funds for the construction of shore and storm protection. Both of these laws have their disadvantages as well as their advantages and it would indeed be advisable if some uniform shore and storm protection legislation could be devised.

Under either law it is necessary for the political subdivision that is issuing the bonds to guarantee the payment of the principal and interest by additional Ad Valorem Taxes, if such should ever become necessary.

Readers are referred to the JOURNAL *for October, 1930, for discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1930.*

Discussion of Paper by McMillan and Lyse:

"Some Permeability Studies of Concrete"*

BY ARTHUR RUETTGERS†

The appearance of the paper "Some Permeability Studies of Concrete" was doubtless welcomed by many students of concrete who had come to recognize the importance of permeability but failed to acquire an intimate knowledge of the subject in its many phases after draining the limited available sources of published information. It must also have pleased the readers to learn that the contents of the paper reflected but the beginning of an extensive investigation by the Research Laboratory of the Portland Cement Association which promises to unearth and dispell many of the mysteries in which permeability of concrete has been shrouded.

The elaborate scale on which the permeability specimens are being tested and the degree of laboratory precision manifested by the general regularity of the data curves thus far presented inspire a feeling of confidence in the test results. One must be careful in interpreting such results, however, to keep in mind the limiting conditions imposed by the particular apparatus and test methods employed.

The paramount role played by early curing in the development of watertightness in concrete, assuming proper placement of the material, seems amply demonstrated by the authors and the supporting test data are especially valuable in view of the previous dearth of reported information along this line. That water content bears a very important relation to permeability seems to have been more generally known and revealed by prior investi-

*By F. R. McMillan and Inge Lyse, Journal of the American Concrete Institute, December 1929—*Proceedings*, Vol. 26, p. 101.

†Engineer, U. S. Bureau of Reclamation, Denver.

gators, although until recently the relation was recognized and expressed in terms of cement content rather than water content.

The very abrupt discontinuance in development of watertightness in thin mortar specimens with cessation in curing and exposure to relatively dry air is significant, for it may indicate what to a large extent takes place on the surface of concrete of any thickness under similar conditions. In this connection it occurs to the writer, since permeability and curing are so intimately associated, that information on the effects of deferred or interrupted early curing might also be illuminating and of great practical value.

In accounting for the diminution in flow from specimens with duration of the tests, it is noted that the authors mention continued hydration and obstruction of the passages with silt or other impurities in the water as causes. As a possible further cause, and particularly as explanatory of the phenomenon represented graphically in Fig. 8 of the author's paper, the writer suggests constriction of the pores through further swelling of the specimens from increased water saturation. In the writer's experience with specimens 4 inches thick and cured 14 days prior to testing, there was little indication of diminution in rate of outflow so long as the pressure prevailing at the beginning of outflow was maintained. In other words the combined resistances to outflow had become practically stabilized.

The apparent relation, for early periods of curing, between compressive strength and watertightness of concrete made with different brands of portland cement, as depicted in Figs. 16 and 17 of the paper, is noteworthy. The maximum divergence in leakage values, about 15 to 1, corresponding to a strength divergence of only about 2 to 1, is very striking. It would be interesting to know the relationship for more advanced periods of curing even though, as indicated by the authors, the actual differences in leakage might be so small as to appear inconsequential from a practical standpoint. It might also be enlightening to supplement the compressive strength-permeability results with data obtained by subjecting the various cements to the sugar-lime solution test recently developed and advocated by Thaddeus Merriman. If Mr. Merriman's contentions are valid it would seem that the use of many incompletely-manufactured

or laitance-producing portland cements now meeting the standard specification requirements must be avoided if strength and watertightness at the usual test ages are to be reliable indices of durability under all ordinary conditions of exposure. Also, if the cementing medium in concrete is to be treated as one of the variables affecting strength and impermeability, there would appear to be opportunity for effecting measurable improvement by the use of appropriate additions to regular portland cement, in the form of material thoroughly incorporated in the manufactured product or added separately as a field operation. In this connection mention is made of the discussion by Richard H. Gaines of the paper "Laws of Proportioning Concrete," by William B. Fuller and Sanford E. Thompson, in the 1907 A. S. C. E. *Transactions*. The discussion is an interesting and skillful presentation of theoretical research studies supported by test data which tend to show that portland cement is rendered far more effective in developing mortar strength and imperviousness at 7 and 28 days; first, by thoroughly mixing a quantity of dry finely-divided colloidal clay with the cement; secondly, by adding a small quantity of suitable electrolyte such as alum to the mixing water; or thirdly, by a combination of the processes mentioned. Some progress seems to have been made along these or similar lines. However, before any such special products or processes might be expected to attain extensive use, it appears there must be reasonable assurance of economy and unimpairment in the durability of the resulting concrete.

Regarding the effect of diatomaceous earth admixture on permeability, experience acquired by the writer through direct contact with work in which a particular brand of diatomaceous silica was used and carefully tested does not bear out the authors' apparent conclusion that the only likely benefit from the use of powdered admixtures in the production of watertight concrete is through elimination of placing defects. Besides improving workability, lessening segregation, and practically paying for itself through increased yield of concrete, the admixture was found consistently to increase the compressive strength and to effect very marked improvement in watertightness over plain concrete of equivalent slumps, in mixtures ranging from about 1 part cement to 7½ parts of dry rodded uncombined aggregate to

that of 1 part cement to 7½ parts of aggregate. The detailed results of a series of permeability tests recently completed and supporting the above assertion are now being compiled by the writer with a view to later publication. Meanwhile, and as the literature on concrete reveals numerous other similar experiences with diatomaceous earth admixture, it is hoped that the authors will find it possible to conduct a more exhaustive series of tests with this material and to investigate at least several of the better brands on the market. In conducting such further tests, it is suggested that cognizance be taken of the fact that one of the primary objects if not the chief reason for using diatomaceous earth admixture is to provide workability in economical mixtures which would otherwise be harsh or border on harshness. It is thought for this reason that the permeability tests should not be restricted to concrete mixtures especially rich in cement or containing perfectly-graded smooth aggregates adequate in fines.

The fact that in the writer's experience no particular benefit in watertightness accrued from the use of 3 per cent of diatomaceous silica in specimens of 1 to 9 mix cured 14 days may serve to partially explain the negative results obtained by the authors with short-cured specimens of 1:2:4 and 1:2½:5 mixes; that is, the pores in all of the specimens were probably too large, due to leanness of mix or short curing, to be effectively obstructed by the small swollen particles of admixture. On this basis it is conceivable that the short-cured but richer 1:1½:3 specimens tested by the authors were benefited by the Celite but the benefit escaped detection since apparently none of the specimens of this mix developed outflow and the testing apparatus was not adapted to measuring the comparative inflows.

Watertightness, impermeability, imperviousness, and like terms are of course only relative, inasmuch as any specimens of concrete can be permeated if sufficient water pressure is available. Supplying definite meaning to the terms consequently requires prescribing certain limiting conditions or providing some standard procedure for testing. The standard method for testing concrete in compression suggests an analogous method for testing permeability, the unit of measure, namely, maximum pressure in pounds per square inch of applied surface, being the same in

both cases. In the compression test, specimens of various standard dimensions are subjected to the force of a plunger moving at a standard rate of speed until the specimens fail, while in the permeability test, water under variable pressure might be applied to one face of specimens of various standard dimensions to produce a standard uniform rate of inflow per unit of area until the specimens were permeated and water commenced to drip or flow from the opposite face. Unfortunately such a permeability test, when manually controlled, requires constant attention and laborious painstaking labor for accurate results, especially when the specimens for test are cured for a period of time in conformity with field practice. Moreover, it appears that automatic operation would require equipment too expensive and mechanically involved for efficient economical service. A substitute method, which seems to lend itself more readily to automatic operation, would be based on reversing the pressure and flow factors. That is, instead of a constant flow and variable ascending pressure, the pressure would be held constant and the variable inflow recorded until leakage developed. With standard pressures and sizes of specimens, the permeability index could then be expressed in terms of rate of inflow for the pressure used or per pound of pressure, at the point of leakage, or it might be feasible to convert the test data, after the relation between pressure and inflow became definitely established, to express the permeability index in terms of the water pressure which would have been required to just produce leakage, as in the first method described. The latter information, accompanied by reliable data on effect of concrete thickness on permeability, would be of particular value in connection with designing concrete mixtures for conduits, dams and other hydraulic structures subjected to relatively large hydrostatic pressures.

Where test specimens are cured in water or soaked in water a sufficient length of time before testing under constant pressure, the absorption and swelling factors are largely eliminated and the inflow-time curves (see figure) appear to assume parabolic form, except in unusual cases where non-uniform internal movement of the water through the pores in a cross section of the specimen

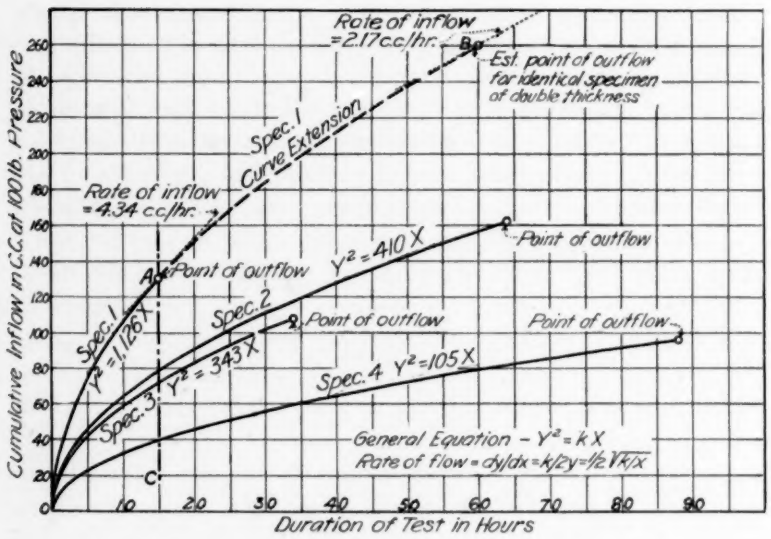

FIG. 1

normal to the direction of flow is prevented by the presence of direct channels which cause premature leakage.

The curves shown in the figure, while not representing actual tests, are nevertheless approximately typical of internal flow curves developed under testing conditions as mentioned. The curves have been drawn as true parabolas and their equations indicated, the ordinates representing cumulative inflow in cubic centimeters at 100 pounds pressure and the abscissas representing the total elapsed hours from the commencement of the test. It is assumed that all specimens represented by the full line curves are identical with respect to dimensions and manner of treatment, except for proportioning the concrete constituents. The general parabolic equation is also stated on the graph, together with the general equation for the rate of inflow. The latter equation, being the first derivative of the general equation for the parabola, is represented graphically by the general tangent to the flow curve.

One purpose of the graph is to demonstrate that if rate of inflow is to be used as a permeability index for comparing the different specimens, the rates should obviously be determined from the ends of the curves or points of outflow, since all specimens, being of the same thickness, then have the same depth of water penetration. Unless this procedure is followed the effect of thickness of specimens is disregarded. The practice employed by some experimenters, while possibly dictated by economic or other considerations, of determining the comparative rates of inflow from some arbitrary period or periods of elapsed time (as along the line AC of the graph) is manifestly incorrect and would in some instances introduce appreciable error. If it is desired to shorten the time required to bring specimens to the point of outflow, higher pressures might be used or reasonably accurate comparative values might be obtained by first determining the approximate voids in the specimens from drying losses or other means and then arriving at the rates of inflow on the basis of cumulative inflow at some fixed proportion of the total void content. The differences in inflow rates obtained from the graphs by using points of outflow as a basis in one case and 15 hours elapsed time in the other case are tabulated below as a matter of information.

Spec. No.	Rate of Inflow in cc/hr.			Apparent Error
	At Point of Outflow	After 15 Hours	Difference	
1	4.34	4.34	0	0%
2	1.27	2.60	1.33	102%
3	1.60	2.40	.80	50%
4	.55	1.31	.76	138%

A second purpose of the graph is to offer some theoretical comments relative to the effect of thickness of specimen on permeability, as this is an important matter apparently surrounded by meagre test data and conflicting conclusions. Referring again to the graph it will be noted that the point of outflow for specimen 1 (point A) corresponds to a cumulative inflow of 130 cubic centimeters. Considering such cumulative inflow as a measure of the volume of water required to saturate the specimen under 100

pounds pressure, it seems logical to assume that twice the amount of water or 260 cubic centimeters would be required to fill the pores of an identical specimen of twice the thickness. Furthermore, and particularly if the extent of curing before test has been such that additional curing action during the test period is not a material factor, it appears reasonable to conclude that the inflow curve for the thicker specimen would be represented by the original curve extended from point A to the point B, at which latter point the rate of inflow (provided the curve is parabolic) would be one-half that at point A. Expressed in general terms, the rate of inflow at saturation, varying inversely as it does with Y, as indicated by the general equation for rate of inflow, would likewise vary inversely with concrete thickness. By similar reasoning it is apparent that the theoretical time required to produce outflow would vary directly as the square of the thickness.

There are other factors affecting permeability on which reliable information is sorely needed, such as gradation and maximum size of aggregates. In the latter connection the writer has found, from direct experience and from plotting test data from various sources, that there is an apparent appreciable increase in permeability with increase in maximum size of aggregate, for mixtures of like water-cement ratio. This tendency might be anticipated in the light of the less effective bond generally secured on the under surfaces of coarse aggregate particles and the increased likelihood for larger pieces of aggregate to collect air, water and laitance by interfering with their rise to the free surface of the concrete.

In planning permeability test apparatus, specimens and procedure, it is believed careful consideration should be given to the facility for practical application of the test data secured. Otherwise it may be found less difficult to secure the data than to convert the results to a form for ready comparison or to apply them to the solution of concrete design problems.

Generally speaking, it appears that permeability knowledge will render material aid in solving two distinct problems of wide import. One is the determination of mixtures and methods required for protecting exterior concrete exposed to the elements and other disintegrating forces. The second problem, peculiar to hydraulic structures, is the determination of mixtures and

methods required for the production of both exterior and interior concrete of sufficient resistance to prevent percolation through the mass, under the particular water pressures to which the concrete will be subjected, or the dissolution, within the mass, of ingredients essential to its durability.

BY GEORGE CONAHEY*

THE AUTHORS of this paper have presented the results of considerable research work studying methods of measuring the permeability of concrete. The results are very interesting.

Before too much value is given to the results obtained, the method they have used should be more fully investigated, and results obtained with a larger number of gradations of aggregate rather than attempt to develop the laws for making of impermeable concrete with the limited number of tests reported and the few gradations of aggregates on which they give results.

The authors of this paper realize this because of the discussion of the different points that they bring up throughout the paper.

Much of the value of their discussion is lost due to the lack of detailed information regarding the specimens tested. For instance, the lengthy discussion in the summary regarding the value of the water-cement ratio appears to apply only to one gradation of aggregate, or at best to aggregate graded to ¾" maximum size. The data on similar specimens in Table II, page 131, show a variation in the permeability of from 50 cc to 1200 cc per sq. ft. per hour with a constant water cement ratio. It is not made clear in the discussion what variables were studied to cause this wide variation with the same water cement ratio.

The following statement is made in the summary on page 142. "Studies of some of the common powdered admixtures showed that additions of those materials which required extra water to maintain plasticity reduced the watertightness . . ." I do not wish to discuss the value of the slump test as a measure of plasticity. Admixtures No. 1, 2, 5 and 6 were tested and placed in this class by the authors. Admixtures No. 1 and 5 are materials purchased as Celite. It is surprising that the authors would

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present such a summary with the limited data available and in the light of their other findings.

The authors state on page 13, "The slight difference in leakages between concrete containing various admixtures and without admixtures must not be given too great emphasis." No doubt this statement is made in view of the limited data available, for they also state, on page 107, "The thickness of the specimen used has generally been 2 in., which gives satisfactory results where the aggregate does not exceed ¾ in. in minimum size." Regarding the tests with admixtures is the statement, "The permeability specimens were 6 x 2-in discs containing aggregate of 1½-in. maximum size." It is not made clear in the paper, but this statement applies only to the data shown in Fig. 18. The use of aggregate graded to 1½-in. maximum size in a 2-in. test specimen is generally considered to be poor practice, and the data obtained from such tests should be classed as "interesting if true" information. The tests made to obtain the data shown in Fig. 19 were made with aggregate graded up to ¾-in. maximum size. These data show no definite trend of results when admixtures were used, and are not sufficient to warrant any statements in regard to the use of admixtures.

The authors point out in the summary, page 140, that "Tests have demonstrated what has been observed in concrete structures that defects in placing are a much more important factor in the leakage than ordinary differences in mixtures, materials, or water content"; also, on page 138, they state as follows: "The concrete must be such that it will readily fill the form and incorporate the reinforcement, and at the same time maintain such plasticity and cohesiveness that segregation will not take place. It is recognized that under certain conditions some of the admixtures may be helpful in producing this desirable consistency and to this extent aid in the production of watertight concrete through the elimination of placing defects."

I should like to point out here that Celite has been successfully used during the past twelve years to aid in obtaining concrete as described above and in correcting the defects often developed in placing. Celite has been used in a long list of jobs where durable, watertight concrete, free from honeycomb, stone pockets and

other defects is desired. The list can begin with the concrete ships built by the United States Shipping Board during the war, and includes such structures as the Cascade Tunnel of the Great Northern Ry., Filtration and Sewage Disposal Plants in various parts of the country, the San Francisco Bay Toll Bridge, the Dam for the City of Altus, Okla., the Tunnel of the Washington Water Power Co. of Lake Chelan, the Bridge of the Richmond, Fredericksburg and Potomac Railroad of Fredericksburg, Va., and the new subway of the City of Philadelphia.

This list could be extended to contain such a number that the editor would not publish it, and the list is not yet completed for Celite is being used in many large construction projects at the present time.

We will follow with interest the future reports of tests made by this method, and also the results of any other tests made to measure the permeability of concrete, for it is a field in which we need considerable more data, and also a more reliable test method, before we can state fully the laws of proportioning and curing concrete in order that it be impermeable. The test method must be such that it will show not only differences between small specimens in the laboratory, but field tests should be made in conjunction with the laboratory tests so that the data can be correlated and future laboratory tests can be correctly interpreted in determining the value of the concrete in the field.

AUTHORS' REPLY TO DISCUSSION BY ARTHUR RUETTGERS AND GEORGE CONAHEY

THE AUTHORS appreciate very much the discussions offered by Mr. Arthur Ruettgers and Mr. George Conahey.

Both these men have taken note of the authors' statement that the investigation is by no means complete and have kept this in mind in presenting their comments.

There are a few places where the text has been misconstrued. It hardly seems necessary to point these out, however, as a careful reading of the paper will make clear the authors' intention.

Both Mr. Ruettgers and Mr. Conahey comment particularly on the data relating to admixtures. Perhaps the authors have not made entirely clear their interpretation of these data as

applied to admixtures. At least they do not find themselves in essential disagreement with either of the two contributors. What seemed to be of particular significance to the authors was the small difference in watertightness between the concretes containing various admixtures and the concretes without admixtures. These were very much smaller than differences due to changes in water content or curing. Thus, it does not appear that *so far as the mixture itself is concerned*, the use of these powdered additions is one of the major factors in producing watertightness. This conclusion is in line with the basic philosophy of concrete mixtures which assumes that any material added to a *plastic* mixture is in no sense a void filler. The material added occupies space just as the other ingredients, and must still be thoroughly surrounded by a watertight paste if the concrete is to be impervious.

This does not assume that the admixtures may not be helpful in producing watertight concrete through their effect on the placeability of the mixture. As pointed out in the paper, the most important factors in producing watertight concrete in a structure are those of proper consistency and details of placing. Any material, therefore, which will insure more satisfactory results in respect to these two factors is deserving of the fullest consideration.

As pointed out in the paper the data reported are the results of the first tests in what is intended to be an exhaustive study of the whole question of durability of concrete. The first attempt was to study those factors which have been shown to be most fundamental in the development of other properties of concrete; namely, the characteristics of the cement, the proportion of cement and water, and the extent of curing. With a clear understanding of the respective functions of the relatively inert aggregates and the active paste, considerable progress has been made in the study of concrete mixtures in relation to the more usual requirements of concrete. It seemed desirable, therefore, to begin this study with an investigation of these same factors to see if the same general principles apply to the development of watertightness and resistance to freezing and thawing. While the data are not complete, even in respect to these factors, it is the belief of the authors that they do establish the application of these principles to a surprising degree.

With a clear understanding of the importance of the quality of the paste and those factors which determine it, the subsequent investigations can be much more accurately controlled and the data obtained more directly applied to the problems of construction. Once the laws governing watertightness of the paste are known it will be very much easier to study such factors as size, grading and characteristics of the aggregate, and methods of placing than would be possible if all these factors were to be studied confused with the further factors of water ratio and curing.

In considering the application of data from tests of this kind to the problems of construction, advantage can be taken of the experience of the past. In a survey of several hundred structures covering a wide range of exposure conditions, as well as such variables as materials, mixtures, and methods of manipulation, it has developed that most of the unsatisfactory concrete could have been eliminated by more care in the selection of consistencies and methods of placing. It is almost universally the case that disintegration is confined to certain sections of the structure and, even in those where the conditions are extremely bad, considerable areas of the exposed surfaces are still in perfectly satisfactory condition. Such trouble as has developed is confined almost exclusively to areas where certain types of defects are to be found. These defects, which are readily recognized upon inspection, could have been easily eliminated by a greater degree of care in certain of the construction operations.

The most common defects are those resulting from the use of over-wet mixes and the practice of allowing concrete to flow over long distances, thus permitting segregation and the formation of deep layers of laitance and pockets of weak, porous concrete. It is quite common to find at the extreme ends of walls, especially in wing walls and buttresses, several feet of concrete in the upper layers which has badly disintegrated, while in the main body of the wall or in the lower layers of the wing walls the concrete is in perfectly good condition.

Another common defect is honeycombing due either to separation of the ingredients from overwet mixes or the use of mixes too dry to compact properly. Still another defect that is universally encountered is the failure to produce proper bond between successive layers or to properly seal the vertical joints between

sections. In defects of this kind water is found to seep through producing unsightly deposits on the exposed face and frequently resulting in disintegration extending well back from the edges.

The fact that in most of the structures encountered, the leakage and disintegration are confined to such points as described is of the most profound significance to the construction industry. That such large areas of the individual structures are still in perfectly satisfactory condition indicates that in the main the materials and mixes were adequate to the exposure had these placing defects been eliminated. To be sure some of the defects of placing are the result of improper mixtures, but a careful study of many structures has led to the conclusion that in a vast majority of cases, even where disintegration is very prominent in spots, only slight changes were needed in the mixtures themselves to have produced thoroughly satisfactory results had the matters of proper consistency and careful placing been attended to.

In the light of these considerations, it must be recognized that these permeability studies take on a new significance. Except in cases of thin sections designed to withstand considerable water pressure, or unusual severity of exposure, the problem seems to be one of ordinary mixtures carefully placed and properly cured. For cases of extreme pressure or severity of exposure, there are precedents to show that excellent results are obtainable with good mixtures properly placed and thoroughly cured. These deductions are in thorough accord with such data as have been obtained from the permeability and freezing tests.

BY G. M. WILLIAMS*

THE QUALITY of impermeability in a concrete is one which has received comparatively little attention in the many laboratory studies which have been carried on to establish the properties of concrete. Yet practically all concrete failures in practice are due to the absorptive and permeable qualities of concrete which may be exposed to changing atmospheric conditions, the leaching effect of pure waters or to salts in solution in ground waters. Structural failures of concrete are extremely rare and inexcusable

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and can usually be traced back to a disregard of well established principles of making, placing and curing concrete. The penetration of moisture followed by freezing or even by alternate wetting and drying has caused marked deterioration of concretes which have apparently been produced with proper materials and methods and which have the requisite structural strength. To insure long life under such conditions of exposure concrete of low permeability must be produced. The writer's experience along this line has indicated that high compressive strength will result in low permeability for any given materials provided consistencies or flowabilities are limited to the minimum suited to the type of work. This relation between impermeability and high compressive strength seems to be as accurate as any of the so-called laws of proportioning concrete for compressive strength.

The importance of low permeability was impressed upon the writer by the results of the Bureau of Standards field investigation of drain tile and concrete exposed to sulphate waters which was started in 1913. Inspection and tests during the following years emphasized that even for the same proportions of cement and aggregate wide differences in resistance to sulphate action could be obtained. Highest resistance was found for those mixtures which were most impermeable. As a result of this experience a permeability apparatus was developed which with minor changes is now in use in the concrete laboratory of the University of Saskatchewan. As shown in Fig. 1 it consists of two circular machined castings with soft rubber gaskets bearing upon the two parallel faces of the concrete test slab. Leakage is prevented by means of a single screw which can be quickly adjusted. Water is applied to the upper horizontal face of the test specimen over a circular area of 28 sq. in. and seepage through the specimen is collected in a glass graduate below. Water under air pressure is held in a tank from which it flows through a pressure reducing valve to the upper face of the specimen. Any number of test specimens can be connected in parallel to the water supply line and if desired additional supply lines and pressure reducing valves can be attached to the tank outlet. Nearly all tests to date have been carried on with a water pressure of 35 lbs. per sq. in. The test specimens are 12 x 12 x 2½ in. molded on edge with the 12 x 12 in. test faces in contact with the sides of the form. In

carrying out a test the specimen after curing or at some later period is placed in position bearing upon the rubber gaskets above and below and the set screw adjusted. After the test is complete the specimen can be as quickly removed and stored away to await future tests.

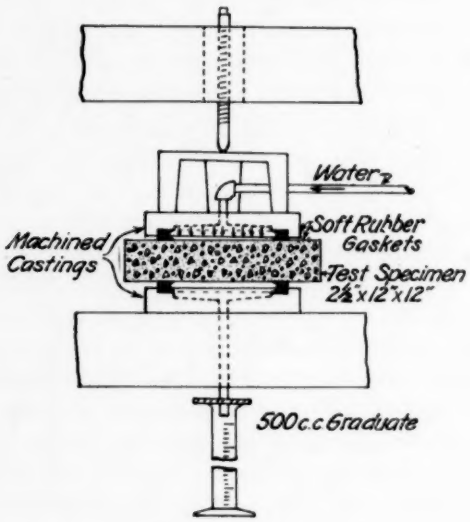

FIG. 1

In discussing the results of permeability tests it will be well to keep in mind the phenomena of expansion and contraction of mortars and concretes when alternately exposed to dry and wet conditions. Many tests have been made to measure the volume change of portland cement mixtures and it has been well established that the drying out of the cement paste causes it to contract, thereby reducing the linear dimensions while the absorption of moisture by a dry or partially dry paste causes an expansion or swelling. The paste is colloidal and the amount of pore space and the size of the interconnecting channels throughout the mass are dependent upon a number of factors, the principal one of which is the moisture content. It is to be expected then that the inherent watertightness of any mixture or its ability to resist flow

of water under pressure is partially a function of the swollen or shrunken condition of the colloidal paste.

The authors state that a 28-day moist curing period at 70° F. for mixtures within the range of commercial practicability resulted in leakage so small that reliable comparisons could not be made between the indifferent variables. Other experimenters have reported the same observation. The writer has encountered the same difficulty and a few years back was preparing concretes with cement contents well below 400 lb. per cu. yd. in order to get comparisons of watertightness. However, it was found that the commercial mixtures after a period of storage in the air of the laboratory became quite permeable, so that comparative results could be obtained in a test in a reasonable duration of time after such storage.

Such results indicate the possibility of several conclusions:

1. The permeability of any concrete is a variable property depending upon the moisture content of the colloidal cement paste.

2. Decrease in leakage or increase in watertightness of concrete during the progress of a test is principally due to swelling of the colloid. This absolute watertightness which is attained when the colloid is fully saturated may be somewhat increased by the further hydration of cement particles and the consequent reactions. For any particular cement some idea of the effect of more rapid and complete hydration may be obtained by comparing the normal cement with the same cement ground finer.

3. For any given water pressure employed in the test a concrete may be reported permeable or impermeable depending upon the moisture content of the colloid.

The variation of permeability has been noted throughout the tests in the writer's laboratory. Specimens after 28 days moist curing may be impermeable for an indefinite time but after storage in air further tests may indicate them to be extremely permeable. The leakage rapidly decreases, just as a dried specimen immersed in water expands very rapidly at first and then more slowly, and approaches constant length. The specimens may then be placed in water for a few days and a further test indicates approximately the same impermeability that existed

immediately after removal from storage. Another drying period in air causes loss of impermeability under the same water pressure.

It would seem that comparable results can be obtained in permeability tests only when the moisture conditions of the colloids are known. Therefore it will be necessary that the specimens be fully dried out or completely saturated just prior to test. Since the most severe condition is the dry condition and concrete in the field may often be in some intermediate condition of dryness the writer believes that tests should normally be made with specimens oven dried to approximately constant weight. It is recognized that the term constant weight is indefinite but a rate of loss at some definite temperature of drying, depending upon the volume of the mass, can be specified. Certainly any dried specimen which is impermeable is inherently impermeable for all practical purposes in the field.

It should be recognized, as the authors point out, that many defects in the field which are evidenced by seepage of water through the mass are often due to conditions which require the placement of large masses with a minimum of spading and tamping. Laboratory studies which result in specifications for a truly impermeable concrete may not necessarily produce such results in the field but proper attention to aggregate grading and the limitation of consistency to that which can be placed with a reasonable amount of tamping and spading will do much to eliminate the defects present in many structures. The use of the old 1:2 sand-gravel ratio together with excess flowabilities is responsible for much of the efflorescence and seepage through weak planes in structures such as dams and reservoirs built of concrete. For any given cement content and aggregate low sanded mixtures tend to be more permeable and even more of the inherent watertightness is lost by the excess water required to float the large volume of coarse aggregate, or due to the accumulated water from lower layers. In many cases where care has been employed and seepage spots have appeared the action probably has been due to the drying out during the curing, and prior to the filling of the reservoir, of a concrete which in comparable laboratory tests would have been impermeable as taken from damp storage but pervious after air drying.

Tests of admixtures by the writer have led to results closely in accord with those reported by the authors. It was also found that the high alumina cements, including the French and American resulted in concretes more permeable than comparable concretes in which portland cements were used.

Where permeability tests have not been made to establish definitely the relation between compressive strength and watertightness for any given cement and aggregate, satisfactory results will probably be obtained by proportioning to obtain a concrete of high compressive strength, avoiding undersanded concretes and limiting consistency to the minimum usable.

Discussion of Hardy Cross' Paper:

"Design of Reinforced Concrete Columns Subject to Flexure"*

BY W. M. DUNAGAN†

When the designer of a structure considers an increase in allowable stresses because of an improvement in the strength of his material he must give equal consideration to the important question of the deformations that will occur under the new stresses. Mr. Cross' analysis of concrete columns subject to fixed deformation promises to bring to the foreground a discussion of the relative elasticity of concrete when increased in strength.

Through heat treatment and special alloys the allowable stresses in steel structures have been increased to a point where the size of members has been reduced in some cases as much as 50%; the extent to which this sort of reduction may be carried in the future may hinge upon this question of relative elasticity. This is true because increases in the strength of steel have not been accompanied by corresponding increases in its stiffness or modulus of elasticity. With this condition obtaining it is conceivable that a structure might be absolutely safe from every point of stress analysis and be unserviceable because of a failure to preserve rigidity. Thus the ultimate criterion in the use of high strength materials may be that of relative stiffness.

Mr. Cross says (page 162), "steel fails at a given stress dependent on its strength whereas concrete fails at a given strain which is practically independent of its strength." He then draws the conclusion "there is no advantage in increasing the strength of concrete which is subject to a fixed deformation." This can only be true if every increase in strength of concrete is accom-

*Journal of the American Concrete Institute, December 1929; *Proceedings*, Vol. 26, p. 157.

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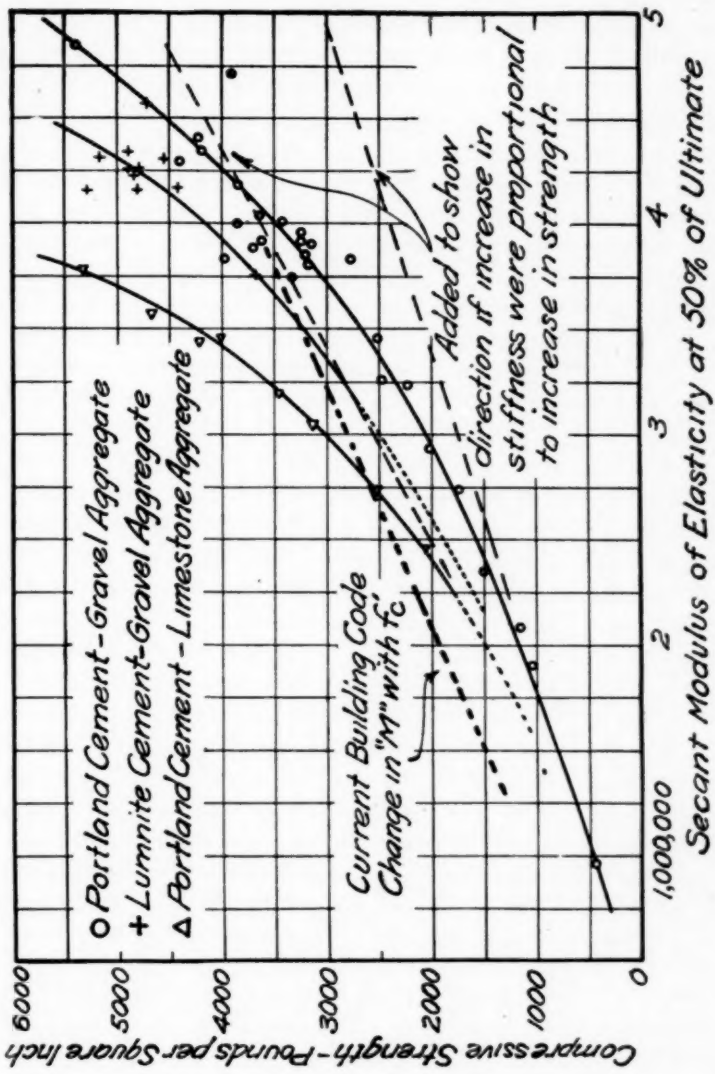
Compressive Strength - Pounds per Square Inch
6000
5000
4000
3000
2000
1000
0
1,000,000
2
3
4
5
Secant Modulus of Elasticity at 50% of Ultimate
O Portland Cement - Gravel Aggregate
+ Lumnite Cement - Gravel Aggregate
Δ Portland Cement - Limestone Aggregate
Current Building Code
Change in "M" with f'c
Added to show direction if increase in stiffness were proportional to increase in strength

FIG. 1

panied by a directly proportional increase in stiffness, which does not seem to be verified in experimental data. Since so much of Mr. Cross' article is based on this conception of the behavior of concrete under stress a careful interpretation should be given the above statements and an effort made to determine the effect of a different behavior on columns subject to fixed deformations.

Fig. 1,* is the results of a study of this very factor; it gives some information as to just how greatly concrete differs from steel when an increase in stiffness is effected with an increase in strength. These data were obtained by means of the mirror-extensometer as described in the Proceedings of the American Society for Testing Materials, vol. 24, part II, Technical Papers, page 1025. The similarity of the curves under the conditions shown and their symmetry vindicates the use of these data for discussion.

To the original curves have been added dotted lines indicating the direction the curves would take if Mr. Cross' conception of the behavior of concrete under stress holds true. Likewise a line has been drawn indicating the direction assumed in the specifications of various building codes when recommending changes in "n" with variations in allowable f'c.

The data on page 161 of the December JOURNAL has been rearranged in order to permit an interpretation of these new data in the light of the figures already given. It will be observed that Mr. Johnson's figure has been plotted in terms of the secant modulus at 50% of the ultimate strength, which fortunately corresponds almost identically with the data with which Mr. Cross derived his figures. To Mr. Cross' tabulations have been added a new set of figures showing the relative deformations that would occur in a column of constant diameter using three classes of concrete each stressed to its allowable f'c.

f'c	Allowable F'c	Relative Strength	Rel. Deformation (New Data)
2000	960	100	100
3000	1290	112	110
4000	1620	119	*119*

*"Relationship between strength and elasticity of concrete in tension and compression," by James W. Johnson, p. 35, Bulletin 90, Eng. Experiment Station, Iowa State College, Ames, Iowa.

From the above data one may be led to say that with each relative increase of strength there is a corresponding increase in the deflection which the member will sustain without rupture; this again is modified by the extent to which "n" is varied with corresponding increases of "E," and as the amount and location of steel is thereby likewise effected.

The writer hopes that this discussion may lead to a closer scrutiny of the entire stress-strain relationship of concrete as it bears on the subject of relative rigidity.

BY STANLEY G. CUTLER*

I was very much interested in the article by Prof. Hardy Cross on the design of "Reinforced Concrete Columns Subject to Flexure."

In this connection we have been designing a large public building in concrete in which the outside wall is unsupported to the height of about forty feet, this wall consisting of large reinforced concrete columns (about 44 in. x 32 in.) with open panels of steel sash between (panels about 18 ft. wide). At the top of the column the roof slab spans across from the outside wall to the interior of this structure.

As the interior of the building will be constantly heated to a uniform temperature while the outside temperature will, of course, vary, it seemed to me that there would be a bending tendency in the columns due to the unequal expansion and contraction caused by this variation in temperature, and the columns would thus have a tendency to curve either in or out. Assuming the more usual case of a lower temperature outside the building than inside, the columns would curve out if fixed at their bases.

If such a column were free at its top no stresses would be caused by this temperature deflection, but it is in fact restrained by the roof slab, which in the above case holds it in at its top and thus sets up a strain in the outside face of the column, causing a stress in the concrete similar to that caused by an outward pressure against the column—i. e., tension at the outside and compression at the inside of the column.

*Olney J. Dean & Co., Chicago.

I did not wish to get into complicated calculations on this account, as the amount of such a temperature deflection would be indefinite at best, so it occurred to me that the equivalent side pressure could be obtained, which would cause a deflection similar to that caused by this temperature strain and its restraint by the roof slab. The restraint at the top of a column would be of the same degree as the deflection in a cantilever under a uniform load, which load was calculated at about 940 pounds per foot of column.

This restraint, however, would not only strain the column but the roof slab as well. This condition I cared for by placing struts in and beneath the roof slab at each column, and running reinforcing continuously from the lower surface of the struts to the outside surface of the columns. The struts were made of such size as to give them approximately the same stiffness as the columns in my judgment, when considered integrally with the roof slab.

The stresses per square inch of concrete from this cause appear to run about as follows: at base of column 580 pounds per square inch compression on the outside; 30 feet up (approximate maximum) 210 pounds per square inch compression on the inside; top of column 150 pounds per square inch compression on the inside; strut at column 150 pounds per square inch compression on top. The corresponding tensions and areas of reinforcing at various sections were similarly obtained. As this method of allowing for temperature deflection seemed to be very liberal, I used allowable stresses in concrete and steel well up toward the elastic limits. The concrete sizes of columns were chosen from these considerations plus the effect of wind and direct load, and then the reinforcing adjusted in like manner. In combining the stresses it seems that the effect of temperature deflection will in general be opposite in effect and somewhat greater than that of wind load.

Upon writing Professor Cross on this matter, he suggested another method of calculation, the result of which would give a fiber stress of about 100 pounds per square inch at the top of the column, but it seems to me that in accordance with practical observation, the critical section on this account would not be at

the top but at some section lower down the column, as appears from the analysis I made.

Probably due to the uncertain rate at which changes of temperature would affect such large columns, any particular refinement in calculation of the stresses set up in them would be superfluous.

However, this type of building is rather common—power houses, assembly halls, theatres, gymnasiums and cell houses, as example, are usually of this general type and it seems to me that if no general calculation of temperature deflection is made for such buildings, at any rate the effect of perhaps 200 pounds per square inch excess concrete stress should be allowed for and the reinforcing arranged to best resist strains from this cause.

This condition seems to me to be specially serious in buildings of this type because if neglected there would be danger of tension cracks on the outside of the columns which would be subject to weathering action, due to the great unsupported height of the columns, and that there would also be likelihood of rather dangerous cracks in the roof slab if the latter was not stiffened to care for the restraining action from this cause.

BY GEORGE H. DELL*

AT THE request of Professor Cross the writer has examined the report of C. Bach and O. Graf in connection with tests upon reinforced-concrete specimens subjected to axial and eccentric loading†, which were performed in 1912 and 1913 at the Kgl. Techn. Hochschule, Stuttgart. This work was planned by Professor Moersch.

The following information deals with a total of 35 reinforced-concrete columns about 8 ft. in length and of cross-sections as shown in Figs. 1, 2, and 3, which were tested to failure under eccentricities ranging from $e/d = 0$ to 1.25, viz.:

Type 1—12 specimens reinforced on tension side with approximately ½ per cent longitudinal steel, without ties;

Type 2—12 specimens reinforced on tension side and on compression side each with approximately ½ per cent longitudinal

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†Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Heft 166 bis 169, 1914.

steel, lateral reinforcement consisting of 7 mm ∅ ties spaced at 7 cm. and a spiral-like reinforcement consisting of 5 mm ∅ spaced at 7 cm;

Type 3—11 specimens reinforced on tension side and on compression side each with 0.95% longitudinal steel and with lateral reinforcement similar to that of type 2.

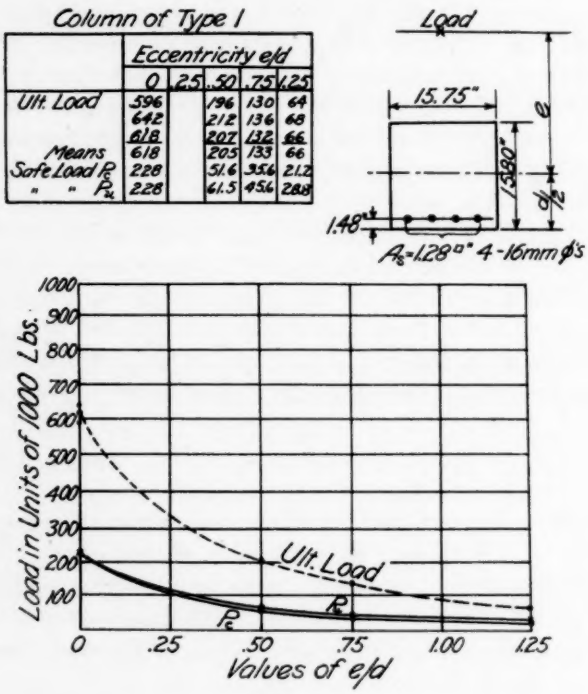

FIG. 1

With regard to the spiral-like reinforcement, three specimens similar to type 2, but having 7 mm ties spaced at 15 cm and no spiral reinforcement were tested,‡ and the average loads, both at appearance of first crack and at failure, when compared with corresponding loads observed on three specimens of type 2 are shown to be within one per cent of the latter. We may then be justified in regarding this type of spiralling, at least in the case of

‡Table 12, loc. cit.

rectangular sections, to be ineffective and in considering that types 2 and 3 are equivalent to plain tied columns.

Materials. The cement had a tensile strength of from 430 to 483 lbs. per sq. in. and a compressive strength of from 5120 to 5330 lbs. per sq. in. (1 day in moist air and 27 days in water);

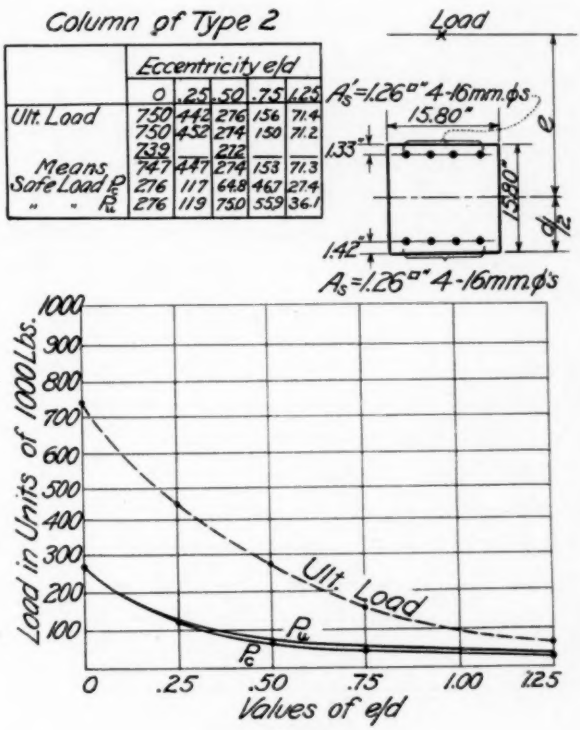

FIG. 2

or, under the new specifications (1 day in moist air, 6 days in water, and 21 days in air), tensile strength, 626 to 769 lbs. per sq. in. and compressive strength 6220 to 6660 lbs. per sq. in.

The concrete consisted, by volume, of 1 part portland cement, 2 parts river sand, and 3 parts river gravel, with 9.2 per cent of water, based on the weight of the dry materials, or an average of 14.3 per cent of the volume of cement, sand and gravel when dry.

The compressive strength of the concrete as found from 53 12-in. cubes at an age of approximately 45 days averaged 3200 lbs. per sq. in., with a maximum deviation of about ± 9½ per cent.

The "upper" elastic limit of the longitudinal reinforcement averaged about 53000 lbs. per sq. in.

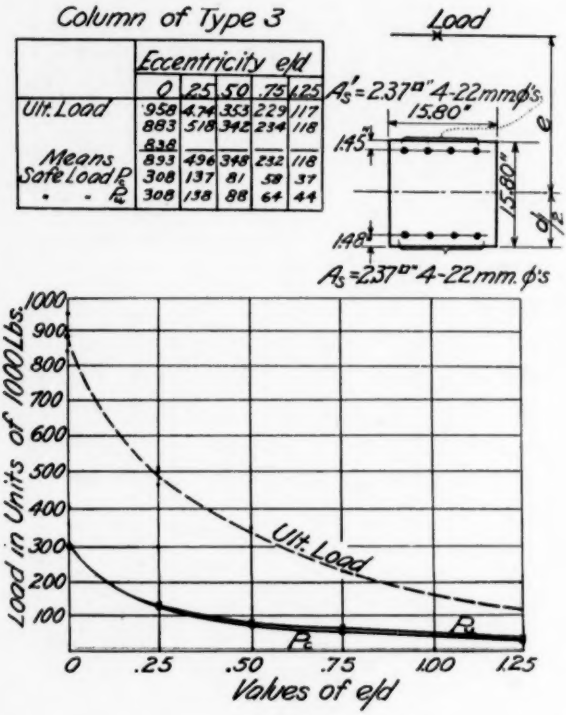

FIG. 3

The specimens were kept covered with wet sacks while in storage; temperature not stated.

Test Results. The curves and tables in Figs. 1, 2, and 3 show the ultimate loads, and the safe loads calculated with n = 15, f_c = 960 lbs. per sq. in. P_c is the calculated safe load obtained

by disregarding the tension in the concrete; P_u that obtained by including the tension in the concrete.

CHARACTER OF FAILURES

Value of e/d	Type 1	Type 2	Type 3
.25		Compression	Compression
.50	Compression	Compression followed by failure of tensile steel.	Compression
.75	Failed almost simultaneously in compression and in tensile steel	Tensile steel, followed by failure in compression	Failure started in compression, followed by failure of tensile steel at ult. load.
1.25	Tensile steel	Tensile steel	Tensile steel

Discussion, including Author's closure, concluded in the JOURNAL *for May 1930.*

Discussion of the Report of Committee 301

Simplified Rigid Frame Design*

BY FRANK J. MC CORMICK†

Professor cross and the other members of Committee 301 are to be commended on the ingenious method of simplified rigid frame design presented in the present paper. There is no doubt that this method may be used to great advantage in many cases. It seems, however, that a few of its limitations should be pointed out a little more clearly.

The author has stated that "the method of analysis proposed here will give accurately the moments at the supports of any frame made up of straight members of uniform section, provided the joints are not displaced by loading . . ." This last clause is particularly significant. But it is also noted that "except for the effect of transverse loads, the joints may be considered fixed in position in practically all cases." This statement is to be questioned. Of course, where frame and load are symmetrical about the same axis the method gives the moments to any desired degree of accuracy when the distributing and balancing operations are repeated a sufficient number of times. In all other cases errors are introduced because there is some unsymmetrical distortion; the amount of error depending upon the unsymmetrical character of the load, or frame, or both.

A simple example will serve to illustrate. In the frame loaded as shown in Fig. 1 the relative stiffness ratios for the left hand column, cross beam, and right hand column are 5, 1, and 1, respectively. The columns are considered as rigidly fixed at their bases. In Fig. 2 the moment curves are plotted for values derived

*Report by Hardy Cross, author-chairman Committee 301, Simplified Rigid Frame Design—Journal of the American Concrete Institute, December, 1929, *Proceedings*, Vol. 26, p. 170.

†Ass't., Agricultural Experiment Station, Iowa State College, Ames, Ia.

by both the slope deflection method* and the approximate method proposed by Professor Cross, the slope deflection method being used as a check as it is generally admitted to be as nearly correct as any of the methods of theoretical analysis.

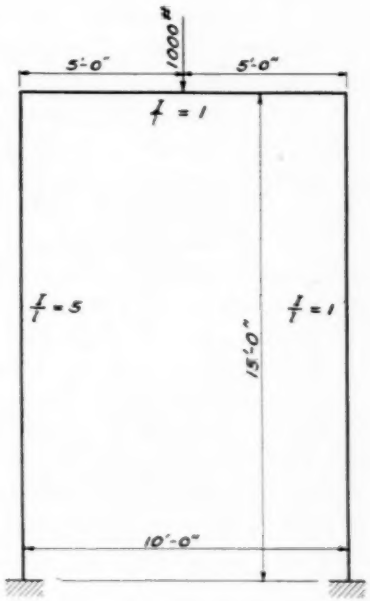

FIG. 1

The solid line enclosing the shaded area is the moment curve found by use of the slope deflection equations. The dotted line is the curve obtained by the author's proposed method. The large discrepancy in the moments at the base of the left hand column is explained by the fact that distortion is not considered in the latter method. For a frame of such proportions even when carrying symmetrical vertical loads, there is an appreciable horizontal deflection to the right which causes the moment to decrease at A and to increase at B. This fact is not considered in the author's proposed method of analysis, and its omission causes us to arrive at untrue values for the moments, particularly

*See Bulletin No. 108, Engineering Experiment Station, University of Illinois, Urbana, Ill.

at the column bases. The largest error in this case is about 355 per cent at the point A, certainly a significant amount.

It would be extremely helpful if Professor Cross could suggest some means for correcting these moments to account for the effect of distortion, particularly lateral deflection.

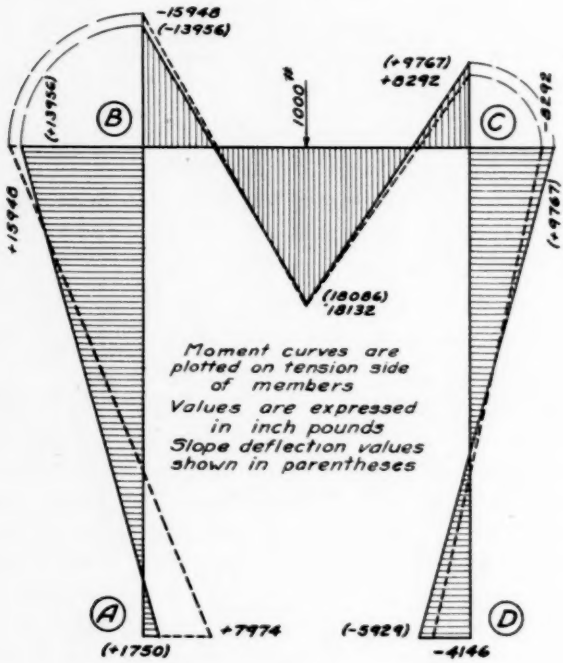

FIG. 2

BY T. F. HICKERSON*

PROFESSOR CROSS' method of computing bending moments at the supports of continuous beams is highly interesting and instructive, and easily understood. But the writer doubts if it is satisfactory for adoption in a design office, because the actual application of the process of so-called *carry-over* moments entails too much re-distributing and juggling of quantities. This pro-

*Professor of Civil Engineering, University of North Carolina.

cedure, if followed through until the dead load and maximum live load moments have been obtained, would become somewhat bewildering when it is realized that these carry-over moments from the first span (of the problem given by the author) must be transferred to the second and thence to the third span; and vice versa. And after this is done, the mid-span moments, the reactions and shears must be found from the equations of statistics.

The writer is convinced that the use of *coefficients* is the simplest and most straightforward method for determining readily the moments and shears at all critical sections. Accordingly, with the expenditure of much time and labor, complete tables of *moment and shear* coefficients, applicable to any case, have been prepared for publication.

By means of these coefficients, one can compute easily the maximum (or minimum) moments and shears at the supports, the quarter point, the middle, and the three-quarter point of each span for all ordinary cases of end and intermediate column restraints.

It is admitted that the proportions of the frame illustrated are unusual for they were purposely so chosen. This was done to emphasize the fact that, while the method proposed by Professor Cross does have many advantages over other methods in a great number of problems, it must be judiciously applied, and is not to be used blindly in all cases merely because of the simplicity of its application.

Discussion of W. M. Dunagan's Paper:

"A Method of Determining the Constituents of Fresh Concrete"*

The importance which is attached to the perfection of reliable means of unscrambling fresh concrete and determining the constituents may be judged by the lively discussion. In his paper in the December 1929 Journal, *Mr. Dunagan mentioned other investigators along similar lines. Their opinions are all represented here in these pages of discussion along with still other contributors. So that the discussion may not be too soon shut off, it is continued to the* Journal *for June, 1930—and should be in the hands of the Secretary not later than May 5.*—Editor.

BY R. L. BERTIN†

The need of a method for determining the constituents of fresh concrete as a measure of uniformity and proportions has been recognized by the writer for a long time, and as Chairman of Sub-Committee XII of Committee C-9 of the A. S. T. M., instigated this test back in 1928 as a basis for writing specifications on mixing, conveying and placing concrete. Various methods of segregating fresh concrete were considered by the committee, and early in 1929 a method was finally developed, a description of which follows:

SUGGESTED METHOD OF DETERMINING QUANTITIES OF MATERIALS IN SAMPLES OF FRESH CONCRETE

1. Prepare a sample of concrete, using sand passing a ¼-in. screen and stone or gravel retained on ¼-in. screen, from which materials smaller than the No. 100 sieve have been removed and which contain water of absorption. Record the weights:

*Journal of the American Concrete Institute, December 1929, *Proceedings*, Vol. 26, p. 202.

†Chief Engineer, White Construction Co., Inc., New York City.

Water = W_1
Cement = W_2
Sand = W_3
Stone = W_4

and total weight of sample = $W_1 + W_2 + W_3 + W_4 = W_5$.

2. Immerse sample in a quantity of water of known volume = V_1 in a graduated container; stir thoroughly to remove all air and observe total volume = V_2.

$V_2 - V_1 = V_3$ = absolute volume of

Water = V_4
Cement = V_5
Sand = V_6
Stone = V_7

contained in the concrete sample.

3. Screen the entire sample through a ¼-in. and a No. 100 sieve, using additional water to thoroughly wash the aggregate free of cement.

4. Collect materials retained on both screens, surface dry, and weigh.

Weight of material retained on No. 4 screen = stone = W_4^1
Weight of material retained on No. 100 screen = sand = W_3^1

5. Immerse stone in known volume of water = V_8 in a graduated container; stir thoroughly to remove all air and observe total volume = V_9

$$V_9 - V_8 = V_7^1$$

6. Immerse sand in known volume of water = V_{10} in a graduated container; stir thoroughly to remove all air and observe total volume = V_{11}.

$$V_{11} - V_{10} = V_6^1$$

7. The absolute volume of water and cement =

$$V_4 + V_5 = V_3 - (V_6^1 + V_7^1) = V_{12}$$

8. Compute weight per cu. ft. of water and cement mixture

$$\frac{W_5 - (W_3^1 + W_4^1)}{V_2 - (V_6^1 + V_7^1)} = \frac{W_5}{V_{12}} = W_7$$

9. Compute water and cement content in mixture weighing W_7 per cubic foot.

Let x = absolute volume of water in a cubic foot of mixture, then $(1 - x)$ = absolute volume of cement

$$62.4x + (1 - x)W_8 = W_7$$
$$62.4x + W_8 - xW_8 = W_7$$
$$x(62.4 - W_8) + W_8 = W_7$$
$$x = \frac{W_7 - W_8}{62.4 - W_8}$$

10. The value W_8 = the cubic foot weight of cement in terms of absolute volume and is obtained from a preliminary test as follows:

(a) Immerse a known weight of cement, say 4 oz. in a known volume of

water, say 200 cc., stir thoroughly to remove all air and observe the total volume of the mixture, say xcc.

$x - 200$ = absolute volume of 4 oz. of cement

and $\dfrac{4 \text{ oz.} \times 1728 \text{ cu. in.} \times 16.3934 \text{ cc. per cu. in.}}{(x - 200) \times 16 \text{ oz.}} = W_8$

11. The absolute volume of cement in sample $= (1 - x)V_{12} = V_5^1$.

12. The volume of water in sample $= x\,V_{12} = V_4^1$.

13. Convert quantities of materials to weight.

$$\begin{aligned} \text{Water } V_4^1 \times 62.4 &= W_1^1 \\ \text{Cement } V_5^1 \times W_8 &= W_2^1 \\ \text{Sand} &= W_3^1 \\ \text{Stone} &= W_4^1 \end{aligned}$$

and compare with original proportions:

$$W_1 - W_2 - W_3 - W_4$$

The above test is based on the assumption that the aggregate does not contain materials finer than the No. 100 sieve.

In practice both the sand and the stone will contain some fines passing the No. 100 sieve which, in the screening process, will register with the cement and water volume and weight and affect the values $W_3^1 - W_4^1 - V_7 - V_8 - V_{12}$. Corrections will therefore have to be made as follows:

14. Determine beforehand the percentage of fines passing the No. 100 sieve for the sand and the stone.

Let p = ratio of fines passing No. 100 sieve to sand retained.
p^1 = ratio of fines passing No. 100 sieve to stone retained.

If the concrete is uniformly mixed, the amount of sand fines passing the No. 100 screen will be pW_3^1, and the stone $p^1W_4^1$.

15. Determine beforehand the weight of these fines per cubic foot in terms of absolute volume in the same manner as described under (10) for cement and let

W_9 = weight of sand fines.
W_{10} = weight of stone fines.

then the absolute volume of these fines will be

$$\frac{p\,W_3^1}{W_9} \text{ for sand}$$

$$\text{and } \frac{p^1\,W_4^1}{W_{10}} \text{ for stone}$$

The absolute volume of water and cement will then be

$$V_4 + V_5 = V_3 - \left\{ V_6^1 + \frac{pW_3^1}{W_9} + V_7^1 + \frac{p^1W_4^1}{W_{10}} \right\} = V_{12}$$

The weight of water and cement will then be

$$W_8 - (W_3^1 + pW_3^1 + W_4^1 + p^1W)_4^1 = W_6$$

The weight per cubic foot of water and cement mixture will then be

$$\frac{W_5 - W_3^1(1+p) + W_4^1(1+p^1)}{V_3 - \left(V_6^1 + V_7^1 + p\frac{W_3^1}{W_9} + p^1\frac{W_4^1}{W_{10}}\right)}$$

The conversion of quantities of materials to weight will then be

Water	$V_4^1 \times 62.4$	=	W_1^1
Cement	$V_5^1 \times W_8$	=	W_2^1
Sand	$W_3^1(1+p)$	=	W_3^1
Stone	$W_4^1(1+p^1)$	=	W_4^1

and these values compared with the original proportions

$$W_1 - W_2 - W_3 - W_4$$

This method was devised with a view of reducing to a minimum the use of predetermined factors because it was felt that this test should be self conclusive.

This method differs from that of Mr. Dunagan in that in the former the apparent specific gravities of the aggregates do not enter in any step of the operation, whereas the latter is dependent entirely upon these values for results and accuracy.

In Mr. Dunagan's method, the amount of water held in the specimen is determined by subtracting from the actual weight of the specimen that of the computed weights of the aggregate and cement. These determinations depend for their accuracy upon the conformity of the apparent specific gravities of the aggregate in the sample to that determined on other samples of aggregate. Recently the Bureau of Public Roads investigated methods of test for apparent specific gravity and absorption of coarse aggregates. The results are contained in a report of D. O. Woolf, entitled "Investigation of Methods of Test for Apparent Specific Gravity and Absorption of Coarse Aggregate," published in the October issue of *Public Roads*. The report indicates that wide variations exist in the absorption and apparent specific gravities of different samples taken from a given material. Table No. 1 is taken from the report and reproduced herein to indicate the variations which may be expected in the apparent specific gravity of aggregates. The variations in percent have been added to the table by the writer to bring out the point in question. The largest variation in apparent specific gravity between the lowest and highest determination of seven samples taken from the same

TABLE 1—VARIATION IN RESULTS OF TESTS FOR SPECIFIC GRAVITY WHEN MADE ON DIFFERENT SAMPLES OF SAME MATERIALS

Sample Number	Specific Gravity Test Number							Average	Maximum Variation between Tests	Variation in % of Lowest Determination	Variation in % above and below Average
	1	2	3	4	5	6	7				
1	2.54	2.58	2.66	2.72	2.66	2.60	2.68	2.63	.18	7.07	± 3.42
2	2.67	2.66	2.67	2.66	2.68	2.68		2.67	.02	.753	± .375
3	2.56	2.64	2.63	2.54	2.64	2.58		2.60	.10	3.93	± 1.925
4	2.54	2.61	2.50	2.56	2.64	2.52	2.59	2.57	.14	5.60	± 2.72
5	2.62	2.53	2.54	2.58	2.58	2.57		2.57	.09	3.56	± 1.75
6	2.60	2.77	2.62	2.68	2.64	2.66		2.66	.17	6.55	± 3.20
7	2.60	2.58	2.61	2.63	2.60	2.55		2.60	.08	3.14	± 1.54

Designation of Materials

Sample No.	Material
1	Massachusetts Gravel
2	New York Limestone
3	Pennsylvania Gravel
4	Ohio Gravel
5	Virginia Gravel
6	Illinois Gravel
7	Alabama Gravel

material is 7.07%)—the average variation for seven different materials taken from various parts of the country is 4.37%—and the average deviation from the average value for each group is 2.13% above and below the average.

If the variations given in Table No. 1 for Illinois Gravel are applied to the test given in Table 1 of Mr. Dunagan's paper, assuming that an apparent specific gravity of 2.60 was found in the preliminary tests and used in the computations and that of the gravel in the sample was 2.70, the computations would result in an error of 15% in the water content and therefore in the water cement ratio.

The test would be inconclusive and misleading inasmuch as the result would be subject to two interpretations.

1. That the W/C ratio of the specimen was as determined, and therefore the concrete represented by the specimen tested unsatisfactory, which of course would be erroneous, or

2. That the W/C ratio of the test specimen was different from that derived from the test, because of variation in the apparent specific gravity of the coarse aggregates, and therefore

that the test was not conclusive evidence that the concrete it represented was defective.

Mr. Dunagan recognizes this point in the "General Notes" of his paper, but the methods of analysis he selected for a general method indicates that he considers aggregates with uncertain apparent specific gravities to be the rare exception, whereas it would appear from the Bureau of Public Roads' report and others to be the rule.

For the reason mentioned above the writer believes that the reliability and accuracy of this test will be greatly improved and its use made more general for all kinds of aggregates if the determinations of the weights and volumes of the fine and coarse

TABLE 2—DETAILED COMPUTATION FOR A TEST

Material Used—Cement
Sand
Gravel

Steps	Sequence of Determinations	Determinations "x" Computations "c"	Designation of Steps	Computations			
a	(1)	x	Weight of concrete sample, in air	3135	3135		
b	(2)	x	Weight of concrete sample, immersed	1848			
c		c	Absolute volume of concrete (a—b)	1287			1287
d	4	x	Weight of surface dried sand	1107			
e	5	x	Weight of surface dried gravel	1377			
f		c	Weight of surface dried aggregate	2484	2484	2484	
g		c	Weight of cement and water paste (a—f)		651		
h	3	x	Weight of sand and gravel immersed			1555	
i		c	Absolute volume of aggregate (f—h)			929	929
j		c	Absolute volume of cement and water paste (c—i)				358
k		c	Sp. Gr. of cement and water paste $\frac{g}{j}$		= 1.82		
l			From Fig. No. 1 cement in paste in %		= 67.00		
m			From Fig. No. 1 $\frac{W}{C}$ ratio		= .755		
n			Proportions by weight—Cement	= g × l	= 436		
o			Water	= g — r	= 215		
p			Sand	= d	= 1107		
q			Gravel	= e	= 1377		

aggregates in a surface dried condition are made and used instead of derived values.

Table No. 2 shows the steps required in carrying out a segregation test, using Mr. Dunagan's method and apparatus with the added steps of weighing the surface dried aggregates and recording the results in the table under Determination 4 and 5. The test thus modified eliminates the necessity of weighing the sand in an immersed condition. the preliminary determinations of the apparent specific gravity of the sand and stone and the computation of the actual weights of the materials. It also removes the element of doubt which must be ever present if the test results are based on questionable premises.

Through the use of a diagram similar to that shown in Fig. 1, the water cement ratio and the amount of cement in percent of weight of cement paste in the specimen may be obtained directly once the specific gravity of the cement paste is known. The diagram is so drawn that if a job curve of water cement ratio strength relations is made, it may be recorded in the lower left hand corner of diagram and the cement paste specific gravity and percentage of cement in the cement paste corresponding to the desired strength or W/C ratio of the concrete read directly.

All known methods of determining the weight and absolute volume of aggregate containing the water of absorption involves surface drying of the aggregate.

The writer is mindful of the time this operation consumes, and that the results of the segregation test would be delayed thereby. It would be, therefore, of great value to develop a method of determining these properties without having recourse to the surface drying process.

Two methods have suggested themselves to the writer, and are given as possibilities.

It is hoped that sufficient interest will be aroused in this problem and that some definite method will be evolved which will make possible immediate determination of the apparent specific gravities of aggregates.

Attention is called to the graph in the lower left-hand corner of Fig. 2. The relation shown is that which concrete strength at 28 days bears to the specific gravity of the cement paste used in the making of the specimen.

It is interesting to note that this relation is practically linear. From it may be deduced the general law that the compressive

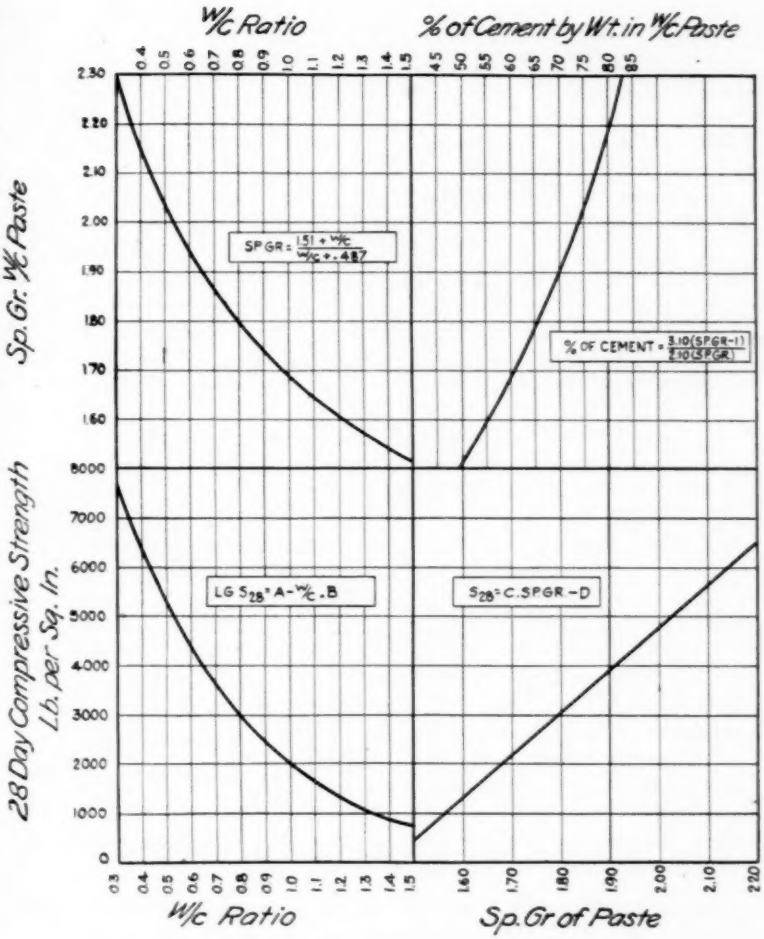
W/c Ratio
% of Cement by Wt. in W/c Paste
Sp. Gr. W/c Paste
28 Day Compressive Strength Lb. per Sq. In.
SPGR = 1.51 + W/c / W/c + .487
% OF CEMENT = 3.10(SPGR-1) / 2.10(SPGR)
LG S28 = A - W/c . B
S28 = C.SPGR - D
W/c Ratio
Sp. Gr of Paste

FIG. 1

resistance of concrete increases within limits in a direct proportion to the degree of uniform concentration of the cement in the paste.

The expression of this relation is in the form

$$S = A \times \text{Sp. Gr.} - B$$

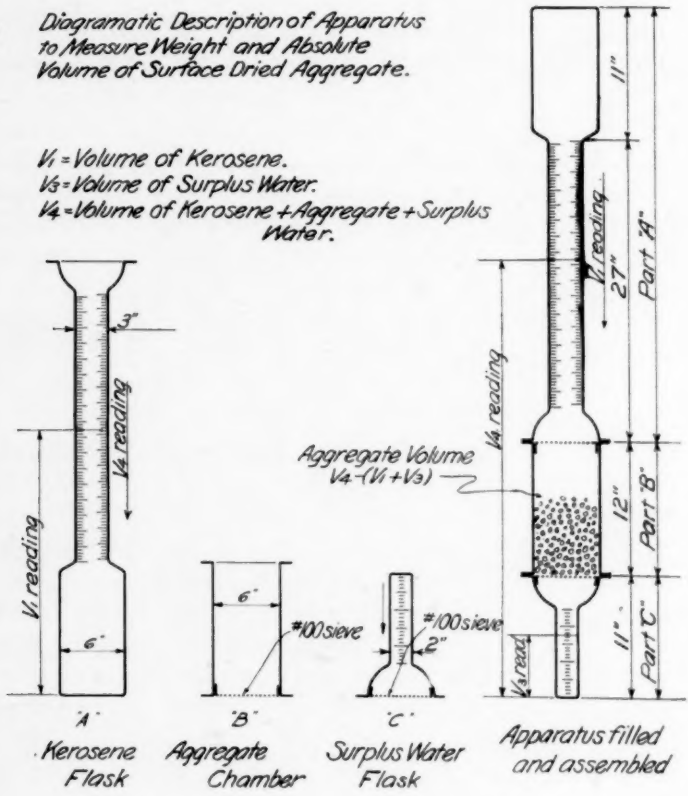

FIG. 2

The writer will publish in the near future a paper dealing with the application of this relation to a number of series of tests conducted at various times by different investigators.

The investigation of existing data by means of this direct relation brings out clearly the simple underlying principle of the W/C ratio law, and throws light on the effects of grading and quantity of aggregate, stiffness of the paste, amount of compaction of the concrete, on the limits within which this basic law applies.

FIRST:—PROPOSED METHOD OF DETERMINING THE WEIGHT AND ABSOLUTE VOLUME OF AGGREGATE CONTAINING THE WATER OF ABSORPTION MEASURED IN A MORE OR LESS SATURATED CONDITION

The apparatus shown in Fig. 2 is devised merely for the purpose of simplifying the description of the method, and for practical use would have to be modified.

The apparatus consists of three parts, namely, part A called the kerosene flask. Part B called the aggregate chamber. Part C called the surplus water flask.

The operation of the apparatus and recording would be as follows: Readings all in grams or cc.

1. Fill the kerosene flask with a measured volume of kerosene = V_1.
2. Weigh entire apparatus with kerosene = W_1.
3. Place the sample of aggregate into the aggregate chamber in the condition it is when removed from the sieves after washing out the cement.
4. Assemble the three parts as shown in the sketch, invert and agitate, water to settle to the bottom of the water flask.
5. Set instrument on scale and register weight = W_2.
6. Read volume of water in surplus water flask = V_3.
7. Read volume of kerosene and aggregate and surface water in kerosene flask = V_4.

From the above determinations, the weight of the aggregate surface dried and the apparent volume may be determined as follows:

Weight of surface dried aggregate = $W_2 - (W_1 - V_3)$

Apparent volume of surface dried aggregate = $V_4 - (V_1 + V_3)$.

The success of this method depends on—

1. The ability of the kerosene to dislodge the surface water from the aggregate.
2. The thoroughness with which the surface water will settle to the bottom of the apparatus.

Other liquids which do not mix with water and having a different specific gravity might be more suitable than kerosene, and in order to facilitate the reading of the line demarkation between the fluid and surplus water, the solution might be colored with a substance which would not affect the water.

SECOND—PROPOSED METHOD OF DETERMINING THE WEIGHT AND ABSOLUTE VOLUME OF AGGREGATE CONTAINING THE WATER OF ABSORPTION MEASURED IN A MORE OR LESS SATURATED CONDITION

1. Weigh sample of aggregate with surface water	=	$W_{a.w}$
2. Immerse sample in known volume		
of saturated salt solution	=	V_s
of known specific gravity	=	G_s
3. Stir aggregate and surface water thoroughly in salt solution and read specific gravity of new solution	=	$G_{s.w}$
4. Weigh sample while immersed in solution	=	$W_{a.s}$
5. Surface water contained in aggregate sample	=	V_w

$$\frac{G_s V_s + V_w}{V_s + V_w} = G_{sw}$$

(a) $$V_w = \frac{(G_s - G_{sw})}{(G_{sw} - 1)} V_s$$

Weight of aggregate containing water of absorption and free of surface water = W_a.

(b) $$W_a = W_{as} - V_w$$

This method employs in part the method of Bell and Collins for finding the free moisture contained in aggregates. This method consists in using a specially graduated hydrometer known as the Collins Hydrometer, which reads directly the amount of surface water contained in a given volume of aggregate. This method is claimed by the manufacturers to be protected by applications for letters patent covering both the method and apparatus.

AUTHOR'S REPLY TO R. L. BERTIN'S DISCUSSION

THE TESTS for determining the constituents of fresh concrete by the writer in the JOURNAL of December, and the one presented by R. L. Bertin in his discussion have much in common; however they differ in one essential respect, that of practicability. In the various steps leading to the development of my test I undertook one on the principle used by Mr. Bertin but could find no means for putting it on a working basis, nor can I find that Mr. Bertin has done so; he cites no examples of actual tests run.

There are two distinct principles made use of in these tests; they may best be illustrated by the fact that they differ in the same respect that the Chapman (A. S. T. M.) Flask differs from the buoyancy method in making moisture determinations on the aggregates. Mr. Bertin attempts to apply the principle of the flask; its weakness being that no refinement for accurate measurement has been developed for anything larger than a 500 gram sample of sand, and, that the moisture in samples must be introduced into the test none of it being lost. In using the displacement by buoyancy principle weighings are made to the nearest one-half gram and only the solid material need be present after initial weighing in air; a condition which is of distinct advantage when making the series of tests to determine the constituents of fresh concrete or for field determinations of free moisture.

The similarity of the two methods and the points of divergency may be noted by comparing each step of Mr. Bertin's method

with corresponding ones in the buoyancy method of Mr. Dunagan. (The reader may refer to Mr. Bertin's discussion for the steps.)

Step No. 1. May be the same for each method.

No. 2. Both methods secure the absolute volume of the sample, the difference being in the choice of method. Bertin secures a direct reading which is dependent on both solid material and moisture present at the time of inundating. Dunagan secures the absolute volume of all material depending upon the solid material present at the time of inundating and of all materials present at the time of weighing initially in air.

No. 3. Same for each method.

No. 4. Dunagan prefers to secure the displacement of the solid material before surface drying to avoid losses in handling. (For any necessity of drying the aggregates, see following discussion of variation in specific gravity). This cannot be done by Mr. Bertin since in the "flask principle" the moisture present from the washing of the cement from the solids will be an unknown quantity.

No. 5. Same for each.

No. 6. Same for each.

No. 7. Bertin secures the absolute volume of the cement and water, Dunagan secures only that of the cement from a difference in that of the entire solids and that of the aggregates, which may at this time be converted to weight in air from a predetermined specific gravity.

No. 8 to 11. Mr. Dunagan eliminates this complicated procedure by a predetermination of the specific gravity of the cement. Bertin's procedure in these steps amounts to the making of a moisture determination on a sample of wet cement as done by the ASTM flask on wet sand except that he relies on differences in previous determinations for his data. His step No. 8 is a determination of the specific gravity of the paste, his step No. 10 that of the specific gravity of cement times 62.4; with these data he could secure the amount of cement and water more quickly by making use of the formula used in moisture determinations:

$$x \text{ equals } 100\,\frac{D\text{-}C}{W\text{-}D};\ \text{Where}$$

a. x equals the percentage of water in the sample.

b. D is W6 in his step No. 10 multiplied by the specific gravity of cement.

c. C is his V12.

d. W is his W6, the weight of the sample.

This would solve for the amount of water in the paste. He could construct a graph such as he has shown in figure (in his discussion) since W7 is the same as step K in his modification of my method.

Steps in making corrections for silt and fine material are but details of the test. The following example illustrates methods of making similar corrections

by the use of my method; they have the advantage that the correction is direct and made from data secured in the same manner as that in which the test for the constituents is performed.

EXAMPLE OF ANALYSIS OF SAMPLE OF FRESH CONCRETE WITH ALL CORRECTIONS

I. Preliminary tests.

a. Take 500 gram sample of cement to be used; weigh this immersed. Pour on No. 100 sieve and wash; thoroughly wash residue back in container and reweigh, thus securing percentage retained on No. 100.

Example: weight of sample immersed 339 grams
weight of part retained immersed 8.5 grams.

Specific gravity equals $\frac{500}{500-339}$ equals 3.10.

Percentage retained on No. 100 equals $\frac{8.5}{339}$ equals 2.5%.

b. Take 1000 grams sample of sand in surface dry condition and weigh immersed. Pour on No. 100 sieve and wash out the silt, put back in container and reweigh.

Example: weight of sample immersed 622.5 grams
weight of (washed) sample immersed 607.0 grams
Loss 15.5

Specific gravity equals $\frac{1000}{1000-622.5}$ equals 2.65.

Percentage passing No. 100 equals $\frac{15.5}{622.5}$ equals 2.5%.

II. Analysis.

	Weights Immersed		Corrected Values			
			Wt. Immersed		Wt. in Air	
Sample in air						3700
Sample immersed	2118					
Coarse aggregate		1150		1150	1845	
Total aggregate	1770					
Sand only (diff)		620	+15*			
			— 9	626	1003	
Cement immersed (diff)	348		— 6*	342	506	
			Total solids in air 3354			—3354
			Water (diff)			346

*.025 × 620 equals 15. and .025 × 348 equals 9. The silt is added to the sand displacement and the cement retained on the No. 100 is subtracted from it. The reverse is true for the correction in the cement quantity.

Water-Cement ratio by weight $\frac{346}{506}$ equals .685.

Water-cement ratio by volume equals 1.025.

Gallons per bag 7.7.

I wish to take issue with the statement that my method is dependent entirely upon previous determinations for its accuracy. The comparison of steps taken clearly shows that this is not so any more than in the Bertin test, but, it presents a method of utilizing them, offering an opportunity for accurate work in a minimum of time. In case of such variations in specific gravity as may occur this method presents a more accurate way to correct for them than does his.

Example: In a given test the stone is of such a nature that a reliable figure for its actual quantity may not be obtained from its buoyancy, the sand is reliable in this respect; in this case the absolute volume of both must be obtained in order to secure that of the cement from their difference with the whole sample, then the stone may be surface dried as a check against the quantity obtained from the conversion from absolute volume to weight in air. The surface drying of coarse aggregate is a relatively quick operation, that of the sand being the more difficult. Mr. Bertin's method *requires* the drying of all aggregates, Dunagan's *permits* the drying of either or both at will without allowing the drying operation to effect the accuracy of the test through losses in handling.

In my general notes I call attention to the fact that coarse aggregates may vary in specific gravity and that they should be watched in this respect. During the development of this device not less than one hundred tests were run at various points in Iowa and Illinois, using material of both quarry and glacial origin; in no case have the results shown such disastrous results as Mr. Bertin prophesies; recently thirty consecutive tests were performed, the aggregates surface dried as a check with no variation of more than 1%.

The committee asks for a "self conclusive" test. How can such exist if the data cited from *Public Roads* are correct? In drying a sample of stone and correcting for absorption a great error is possible since it shows corresponding variations in absorp-

tion with the specific gravity variations. It seems that such data indicate that under certain conditions there is a limit to which the accuracy of control tests may be performed in the effort to produce uniform concrete. Fig. 1, a special test of uniformity in

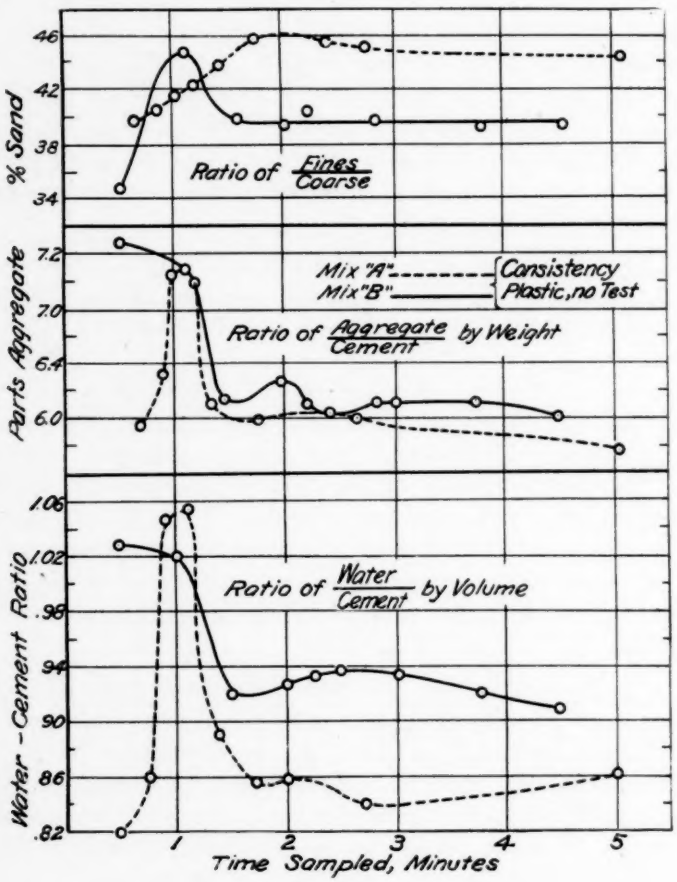

FIG. 1

mixing, in which my device was used to analyze the mix at various intervals of mixing, shows test results to a degree of accuracy as fine as present practice requires for water-cement ratio control of quality.

The data cited by Mr. Bertin on variations in specific gravity must be accepted as pertinent with reservations for the following reasons:

a. The tests were not run under the conditions obtaining in concrete work. He chose samples rather than selecting them from quartered samples of well mixed stock piles, such as occur in concrete work.

b. The method of test employed is one over which the operator has less control than with other methods. A device such as the buoyancy method for running these tests is subject to less error on the part of the operator.

c. Other tests show contrary results. The data from the Iowa Highway Commission laboratories presented by Mr. Meyers, materials engineer, show results from more representative conditions. The Portland Cement Association Laboratory in Chicago ran specific gravity determinations on Illinois gravel as a routine test until it became evident that such consistent uniformity existed that they were discontinued.

d. There are too few tests for conclusive evidence.

Objections have been raised that the weighing of the sample in water might be effected by the trapped air, or air voids. There is nothing to prevent the tester from stirring the sample into the bucket container, in fact it is ideally built for this procedure, and dry samples demand this stirring with one minute for settlement.

The example cited in the December JOURNAL was used for the specific reason that it shows that the test is more accurate for determining the water-cement ratio used than for the other constituents. The difficulty in securing a sample respresentative of all of the constituents is obvious, however if through specifications the net water is to govern the quality of the concrete, this test nicely eliminates a lot of worry about them. In fact the screening out of the large aggregate and testing of only a sample of the mortar for its water and cement will be sufficient unless the cement-aggregate relations are desired. In case they are desired, a sample of not less than 10 pounds should be selected and weighed immersed; this may then be sieved in sections and the test run exactly as shown for the smaller ones in the examples cited in this paper.

In reply to many letters of inquiry concerning the complete

TABLE 1—RESULTS OF ROUTINE TESTS ON CHECK SAMPLES OF CRUSHED LIMESTONE SENT TO THE AMES LABORATORY DURING CONSTRUCTION SEASON, 1929

Location of Plant	Specific Gravity			Average Variation from Average Specific Gravity	
	Max.	Min.	Aver.		No. of Tests
Glory, Iowa	2.67	2.57	2.62	.015	13
Lidwood, Iowa	2.68	2.64	2.66	.011	15
Buffalo, Iowa	2.69	2.65	2.67	.010	4
Dubuque, Iowa	2.70	2.65	2.68	.013	12
Dubuque, Iowa	2.69	2.63	2.66	.011	18
Des Moines, Iowa	2.60	2.55	2.58	.016	10
Alden, Iowa	2.61	2.48	2.55	.027	31
Le Grand, Iowa	2.60	2.54	2.57	.020	4
Lidwood, Iowa	2.68	2.60	2.64	.025	14
Sandusky, Iowa	2.60	2.51	2.56	.021	12
Le Grand, Iowa	2.63	2.49	2.56	.032	17
Gilmore City, Iowa	2.67	2.60	2.64	.021	8
Mason City, Iowa	2.69	2.64	2.67	.009	10
Coralville, Iowa	2.69	2.63	2.66	.020	9
Marquette, Iowa	2.70	2.65	2.68	.023	3
Croton, Iowa	2.66	2.65	2.65	.002	4
Fayette, Iowa	2.67	2.61	2.64	.018	11

TABLE 2—RESULTS OF ROUTINE TESTS ON CHECK SAMPLES OF GRAVEL SENT TO THE AMES LABORATORY DURING CONSTRUCTION SEASON, 1929

Location of Plant	Specific Gravity			Average Variation from Average Specific Gravity	
	Max.	Min.	Aver.		No. of Tests
Bellevue, Iowa	2.67	2.62	2.65	.013	19
Camanche, Iowa	2.65	2.60	2.63	.011	20
Des Moines, Iowa	2.72	2.67	2.70	.020	6
Clear Lake, Iowa	2.69	2.64	2.67	.008	8
Clarksville, Iowa	2.72	2.61	2.67	.020	25
Eddyville, Iowa	2.68	2.62	2.65	.018	13
Gilbertville, Iowa	2.71	2.61	2.66	.032	8
Des Moines, Iowa	2.69	2.62	2.66	.022	4
Mason City, Iowa	2.72	2.62	2.67	.019	8
Des Moines, Iowa	2.69	2.61	2.65	.040	2
Sacton, Iowa	2.69	2.60	2.65	.018	13
Ottawa, Illinois	2.71	2.63	2.67	.021	9
Lake View, Iowa	2.72	2.63	2.68	.021	18
Shellrock, Iowa	2.67	2.62	2.65	.015	11
Winona, Minnesota	2.69	2.65	2.67	.027	3

equipment with which this test is performed the following information is given. The apparatus was developed for field or laboratory use in performing:

1. Approximate apparent specific gravity of fine and coarse aggregates and portland cement.
2. Free moisture or absorption of fine and coarse aggregates.
3. Silt Determinations.
4. The analysis of the constituents of fresh concrete.

Tables I and II are the tabulations of the results of routine tests on samples of the different concrete aggregates sent to the Ames Laboratory during the construction season of 1929. The samples were sent in weekly by materials inspectors stationed at the several material plants producing material for use in highway work. Tests were performed in the ordinary routine of the laboratory using the jar pycnometer, no special attention being given to any of the tests performed other than to maintain the uniformity of practice required by standards set by that laboratory.

These tests were performed by different operators on different samples of the same material. Consequently the variations obtained may be due either to the relative inaccuracy of the different operators or to the variations in the characteristics of the material itself; there is no means of differentiating the two sources in this case. However it will be observed that such variations as are shown in these data are much less even though caused by both the material and the operator than those shown by Mr. Woolf and attributed to the material alone.

BY C. A. HUGHES*

THE WRITER agrees with Professor Dunagan that determination of the composition of freshly placed concrete is the most direct, and therefore the most satisfactory method of checking the efficiency of mixing and placing operations.

The requirements of test procedure for this purpose are that (1) accurate and dependable data are obtained, (2) the test may be completed rapidly, and (3) the apparatus is suitable for field use. The proposed method appears to satisfy these requirements.

Assuming workable concrete and acceptance of the water-cement ratio law, such a test can leave little room for argument as to the inherent quality of the mix. Due to the short time required to perform the test, corrections indicated by the test data can be

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applied before any great mass of concrete is placed. Its application would be of special value in quantitatively expressing the degree of uniformity obtained.

About two years ago the writer had occasion to make some analyses of the composition of the concrete used in laying a wearing course in a large commercial building. While a different (and far more arduous) method was used, the data obtained may be of interest as an application of the general principle.

The wearing course was laid by a patented process consisting essentially of placing a fluid mix, reducing the water content by an absorptive mat, and then mechanically consolidating the dried mix. Measurement of aggregate was by wheel barrows and water was added by pail. The mix was stated to be 1:1:2 by volume, the aggregates being washed sand and pea gravel.

Samples were taken by placing flat galvanized pans side by side on the structural slab, the concrete being deposited over them in the usual fashion. One pan was lifted immediately after screeding and the second after the removal of the absorptive mat. Three samples each of the wet and dry mixes were taken per day for four days, the interval between sampling being about one hour.

The concrete in each pan was thoroughly mixed and quartered to give two samples, each weighing 1000 g. One of these samples was used to determine the water content and the other to determine the cement content.

The water content was obtained by drying to constant weight, the quantity so obtained being corrected by the amount absorbed by the aggregates to give the effective water used in the calculation of the water-cement ratios.

The cement content was obtained by decantation, all washings being made by one man who was most particular in following the specified procedure to the letter. Laboratory tests of samples similar to those tested on the job but containing known quantities of cement showed that not all the cement was removed in this way, and that the per cent removed was somewhat different for the two brands of cement used. The method of calculating the quantity of cement can best be expressed algebraically in the following equation:

$$C = \frac{W_1 - (W_2 + W_3)}{p}$$

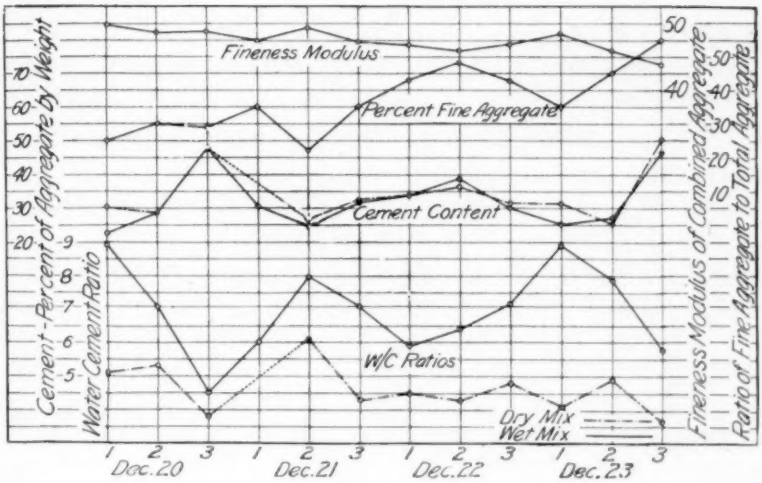

FIG. 1

in which C = cement in grams
W_1 = weight of sample as taken
W_2 = dry weight of sample after washing
W_3 = weight of water removed from companion specimen
p = ratio of cement removed to total cement as determined by the laboratory tests.

Sieve analyses were made on the materials from the stock piles and on the washed samples of the wet mixes. From these data, the proportions of fine aggregate to total aggregate were calculated from the respective fineness moduli.

The data obtained are given graphically in Fig. 1. The horizontal scale is arbitrary and of no significance. The two lower graphs show the water-cement ratios for the mixes as placed, referred to as wet mixes, and those for the mixes after application of the absorptive mat, referred to as dry mixes. The variation in w/c ratio for the wet samples is marked, having as a maximum .89 and as a minimum .45. The low value for the 3rd sample of Dec. 20, appears to be abnormal and to result from the high cement content. Eight of the twelve determinations are between .8 and .6. The w/c ratios for the dry mixes are much more uni-

form. The No. 2 specimen of Dec. 21 was taken from an area that was found to be too wet after the initial drying. Subsequent to taking this sample, the area was re-dried for a period of 40 minutes before finishing. With this exception, the w/c ratios for the dry mixes lie approximately between .5 and .4.

Assuming that no change in cement content results from the application of the drier and that the mix is homogeneous throughout the quantity required to cover the two adjacent sampling pans, then the percentage of cement determined from the wet samples should equal that determined from the dry samples. The three largest differences are 7.9% for No. 1 spec., Dec. 20, 6.4% for No. 1 spec. and 3.7% for the No. 3 specimen, Dec. 23. It is likely that the determination for the No. 1 specimen of Dec. 20, is in error since this was the first one taken. The mean variations for all determinations including the first is 2.45%. Neglecting the first, it is 1.91% or expressed as a percentage of the cement content about 7%. The 6.4 variation expressed as a percentage of the cement content of the wet sample is 25%. It will be noted from the graphs that in the eleven double determinations the cement content for the dry mix is higher than that for the wet mix in 6 instances, equal in 3 and less in 2. This suggests that additional cement is added to the mix by the application of the drier. Observation of the process would also lead to this conclusion. Considering the possibilities of error and that the determinations were made in the field, the results are felt to be satisfactorily accurate.

For a 1:1:2 mix by volume the calculated quantity of cement expressed as a percentage by weight of the total aggregate is 29.5% and approximately 33 per cent when bulking due to moisture and loose measurement is taken into account.

Abrasion tests and visual inspection of the wearing course showed that a very satisfactory surface was obtained, as would be expected from the low water-cement ratios of the dry mixes. Had Professor Dunagan's method been available at the time these tests were made, the same data could have been obtained with much less labor.

BY H. C. ROSS*

IN THE past year experiments on the analysis of freshly mixed concrete were carried out in the laboratories of the Hydro Electric Power Commission of Ontario.

Two series of tests were made using materials sold commercially in Toronto. In the first series the samples were of such a size as to be within the capacity of the scales and graduates used in the laboratory and the method of test employed was that which has been given in Mr. Bertin's discussion of Professor Dunagan's paper. The results of these experiments are shown in the accompanying table. The second series of tests used larger samples and duplicated as nearly as possible conditions and equipment which would be encountered in the field.

In the second series of tests the samples were large enough to be considered representative of the concrete taken from the mixer or from the forms. The total volume of concrete was found by immersing the sample in a large container of water, fitted with an overflow tube inserted in the side. The mixture was thoroughly agitated to remove the air, and the overflow water allowed to collect in a separate container. The volume of this water, calculated from its unit weight gave the total volume of water in cubic feet. The rest of the experiment was carried out using the same procedure employed in the first series, except that the volumes of sand and stone were calculated from their dry weights and apparent specific gravities rather than by actual determinations. It was assumed that the specific gravities would not vary to any appreciable extent from day to day. The results obtained from these tests are shown with those of the first series.

The results obtained in the second series approach closer to the actual proportions than do those of the first series, but it is probable that if larger samples had been used in the first series an equal degree of accuracy would have been obtained.

The greatest objection to this method of determining the constituents of freshly mixed concrete is the time required to make a complete determination. In the second series, using samples weighing about eighteen pounds, the time required to make the test and to complete the calculations amounted to about two

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SEGREGATION TEST ON FRESHLY MIXED CONCRETE

First Series	Original Proportions	Experimental Determinations (1)	(2)	(3)	(4)	(5)
Water, grams	26.8	30.3	25.2	26.2	24.4	25.5
Cement, grams	44.8	41.4	47.0	44.0	49.5	45.9
Sand, grams	128.0	127.35	126.74	129.37	127.17	128.06
Stone, grams	192.0	192.6	192.7	192.2	190.5	192.2
Prop. by vol	1:2.3:3.8	1:2.5:4.1	1:2.2:3.6	1:2.4:3.9	1:2.1:3.5	1:2.3:3.7
w/c ratio by wt.	.60	.73	.54	.60	.49	.56
Second Series						
Water, ounces	20.2	21.6	19.9	18.0	20.1	20.4
Cement, ounces	33.6	33.1	33.4	35.7	34.2	34.6
Sand, ounces	96.0	98.01	98.01	99.02	96.5	96.0
Stone, ounces	144.0	144.04	142.04	141.04	143.05	142.79
Prop. by vol	1:2.3:3.8	1:2.4:3.8	1:2.4:3.8	1:2.3:3.5	1:2.3:3.7	1:2.3:3.7
W/C ratio by wt.	.60	.65	.60	.50	.59	.59

and one-half hours. A large portion of the time was consumed in drying the aggregate after washing through the sieves.

Although the results show that an approximate check, and at times an accurate check on the actual constituents of the mix can be obtained, they also show the possibility of wide variations in water-cement ratios which would give misleading information regarding the actual proportions of the mix.

The accuracy of both Professor Dunagan's method and of the second series of the Commission's tests depends to a large extent upon the accuracy of the figures for specific gravity. Although our experience leads us to believe that for a given material the specific gravity would not vary sufficiently to cause serious errors, yet the possibility of such variations would make the use of this method somewhat hazardous.

All of the methods proposed determine the amounts of water and cement by first finding the quantities of sand and stone either by calculation or by actual determination, and then subtracting these qualities from the total weight or volume of the sample. This means that all errors from these calculations or determinations will accumulate and appear in the figures for water and cement, the most important constituents of the mix. It is a question whether any method in which the determination of the water-cement ratio depends upon the degree of accuracy with which the rest of the constituents are determined will be found

satisfactory, and could be considered a dependable check on the actual proportions of the mix.

BY GEORGE J. GRIESENAUER*

I AM at a disadvantage in offering discussion of Mr. Dunagan's paper as I have not had the time nor are we equipped with the apparatus described to make the test. Mr. Dunagan, I believe, was with the group of men in the Portland Cement Association laboratory when I made a demonstration test of my method. He is probably more familiar with both methods than I am and could rate their comparative values.

Mr. Dunagan states that Mr. R. L. Bertin of New York and Mr. W. I. Freel of Purdue University, have also worked on a method for the same purpose, namely to determine the amount of the different constituents in a sample of plastic concrete, especially the water and cement content. Their methods make use of the specific gravity of the different materials in the sample and from these the quantities of the different materials are found by calculation.

When it is remembered that all the equipment that is required to make the analysis by my method are No. 100 and No. 4 sieves, a scale, a wash basin, a pail and a gas or electric plate, and that all of the many tests I have made have been made under crude arrangements, its simplicity is evident. And further, when the simplicity of the mathematics involved to obtain the results is realized, as indicated in my article in the Engineering News-Record of Nov. 28, 1928, this method becomes attractive indeed.

I would not disparage any of these more recent investigations. Now that a way is known for unscrambling plastic concrete to determine its quality immediately instead of waiting for the compression test for that information, I welcome the thought and attention that is being given the subject. The idea, it seems to me, is so valuable that it is worth all of the thought and work necessary to make it so simple and practicable that we cannot afford not to use it.

My method for making an analysis of plastic concrete, after obtaining a representative sample of about 10 to 15 pounds of the newly made concrete which has been put into a tightly sealed

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receptacle (a friction top can is a convenient container) and set aside until ready to make the test is as follows:

Weigh the package (sample and container) transfer the sample into a pail of about 3 gallon capacity, add water, stir and decant into another pail of about same size; repeat this washing until the cement is all removed from the sand and gravel. Weigh the empty container. Let the pail containing the decantation stand for a few minutes; remove the sand and gravel from the pail onto a pad of several layers of newspaper; change the sand and gravel occasionally onto dry pads of newspaper to expedite the drying.

Pour off the clear water from pail containing the cement. Set pail on gas plate burner to dry, cool and remove the cement from pail onto a pad of paper and pulverize with a round glass jar, roller pin fashion, and sieve through a No. 200 sieve. Repeat pulverizing the part remaining on the sieve until all lumps are pulverized, sieve and return residue on sieve to the sand and gravel.

Screen the sand and gravel on a No. 4 screen, weigh as gravel the part remaining on screen. Sieve the sand on a No. 200 sieve. Add the amount which passed through sieve to the cement. Weigh as sand the amount retained on the sieve. Weigh the cement.

From the net weight of the original sample subtract the combined weight of the cement, sand and gravel. The difference represents the amount of water the sample contained.

Use the net weight of a sack of cement (94 lb.) to determine the factor to use to find the weight, in pounds, of water used per sack of cement. For example: Cement amounts to 2.0 lb.; water to 1.25 lb. Divide 94 by 2.0 equals 47.0, which is the factor to use. 47 x 1.25 equals 58.75 pounds of water, divided by 8.33 equals 7.05 gallons of water per sack of cement.

By referring to Abrams curve we find that 7.05 gallons of water per sack of cement produces concrete of 2,375 lb. at 28 days old.

For computing the amount of sand and gravel, the factor 47 is likewise used.

From the above it will be seen that the only apparatus required to make this test is a scale, a gas plate burner, No. 200 and No. 4 sieves and a couple of 3 gallon pails.

The time required to make the test depends on the time it takes to dry the materials, pulverize the cement and screen the materials. Under favorable conditions this should require less than an hour. I have not attempted to determine the minimum time this analysis could be made in. It has been my practice to do this work after supper in our basement and I have worked at it leisurely because I have enjoyed this work of unscrambling plastic concrete.

BY W. I. FREEL*

THE METHOD of determining the constituents of fresh concrete mixtures as outlined in the December issue of the JOURNAL is a very ingenious one. It eliminates the difficulties met by the author in seeking a practical solution to the problem.

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Mr. Dunagan says, "The resulting concrete has depended largely upon the skill and determination of the engineer in the execution of each of the above operations. Up to date he has had no way of directly checking the character or uniformity of his product, as placed in the forms, except to await the results of strength tests at some later period." It is the belief of the author that it is not yet time to discard the strength tests usually made as a verification of the quality of the concrete placed on the job. Again Mr. Dunagan says, "The inspector need not concern himself with volume measurement or moisture determination, nor any of the details of how the most suitable mixture is obtained, but need only determine that the specified concrete had been placed in the forms and that correct curing procedure follows." It will be some time before the details of obtaining the most suitable concrete mixture can be trusted to any except the best equipped and most conscientious of builders.

Probably the best use of the method as outlined will be in securing data for the purpose of discovering facts about methods at present practiced in proportioning, mixing, transporting and placing concrete mixtures.

During the spring of 1929 the following method of analyzing fresh concrete mixtures was outlined by the author in connection with the preparation of a graduate thesis under the direction of Dr. W. K. Hatt of Purdue University.

The underlying principles of the method were pointed out by R. L. Bertin of the White Construction Co. in letters written to Dr. W. K. Hatt during the progress of the preparation of the above thesis.

FREEL'S METHOD OF ANALYZING A FRESH CONCRETE MIXTURE

The method outlined below requires the use of a cylinder, the minimum capacity of which should be one-fourth of a cubic foot. The cylinder should be of steel of such a gage that rough handling will not distort its shape. The top should be faced off smooth enough for a piece of plate glass, with diameter slightly greater than the outside diameter of the cylinder, placed on the open end to have contact entirely around its circumference. It is necessary to have a weighing scale reading to one-tenth of a pound with a capacity consistent with that degree of accuracy.

The method of test is as follows:

Take a sample of the concrete from the form and puddle it into the container until the top surface of the concrete paste is above the faced rim. Force the

glass plate down on the concrete mixture in such a way as to exclude the air and excess concrete and weigh after the container is cleaned off on the outside.

This measurement makes it possible for us to calculate the net weight of the known volume of the concrete in the cylinder. The sample is then washed over a No. 4 sieve and a No. 100 sieve in such a way as not to lose any of the gravel or sand. This washing must be very carefully done. Of course, in washing the sand over the No. 100 sieve nearly all of the cement and some of the sand are washed through. After washing the aggregates the sand is placed into the container again, care being taken not to lose any of it. The measure is filled about two-thirds full of water and carefully stirred to remove

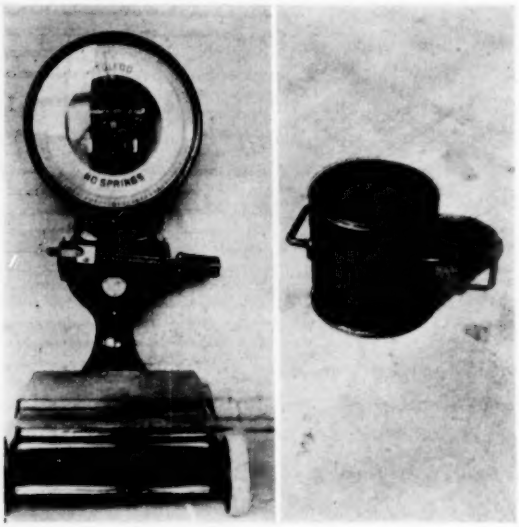

FIG. 1

the air. (There is considerable quantity of air entrained, even though the sand is wet). After removing the air, the cylinder is placed on a horizontal surface and filled with water until the meniscus has the desired convexity. The plate glass is again placed firmly on the cylinder, the cylinder wiped and weighed. This weight is the key to the absolute volume of sand retained on a No. 100 sieve. It may be explained in this way: Whenever one pound of water is displaced by the absolute volume of sand equivalent to the volume occupied by one pound of water, the total weight of the container, sand, and water is increased by a weight in pounds equal to the specific gravity of the sand minus one. Without removing the sand the coarse aggregate is added to the contents of the cylinder. (Not much difficulty is experienced with the air

clinging to the coarse aggregate). The measure again placed on a horizontal surface and the glass plate pushed down on the convex meniscus. The weight of the cylinder and its contents is taken again. Of course, in this case as in the previous one, the volume of the aggregate added can be calculated from the weight change and the specific gravity of the gravel.

Having the volume of the concrete, the volume of the find sand, and the volume of the coarse aggregate (absolute volumes), we may find the volume of the cement water paste. Knowing the weight of the gravel, the weight of the sand, and the weight of the concrete, we can find by subtraction, the weight of the cement water paste. Knowing the weight and volume of the paste in the mixture and knowing the specific gravity of the cement to be equal to 3.1 (approximately), we can readily see how much cement and how much water there was originally in the mixture. One thing has been left out, that is, the consideration of the part of the sand lost in washing the sand over a 100 mesh screen.

It is necessary in using this method of analysis to be able to have a sample of the sand used in the mixture. This sample is dried to constant weight and passed over a 100 mesh screen. That part of the sand retained is weighed and that part that passes is weighed. The weight of the sand passing the 100 mesh screen is expressed as a ratio to the weight of sand retained. Before subtracting the volume of the sand from the volume of the container and before subtracting the weight of the sand from the weight of the container, the volume and weight of the sand respectively should be multiplied by this ratio.

It is necessary to perform a few auxiliary tests to aid us in our calculations.

A sample of moist sand taken from the stock pile should be surface dried and the apparent specific gravity of it in that state determined by the use of the method outlined in "Tentative Method of Test for Field Determination of Approximate Apparent Specific Gravity of Fine Aggregate. Serial Designation: C68-27T, A. S. T. M."

From the gravel in the stock pile take a moist sample somewhat greater than one thousand grams, dry each piece with a damp cloth and find the apparent specific gravity of the surface dry gravel by measuring its displacement.

This completes the tests necessary to the performance of the analysis.

Of course, in using this method the results obtained must be assumed to be reasonably correct, for there is no way of checking them. The sample is only a very small part of the whole batch of concrete. A rough check on the constitution of the original batch may be obtained by reference to the proportions of the batch sampled. Of course, a single test on a batch of concrete is not enough. It will be necessary for us to obtain a sample from at least two parts of the batch.

It is to be remembered that in measuring the ingredients of the mixture, weighing scales accurate to the nearest one-tenth of a pound are used. This is about as accurate a measurement as

one can expect to make in ordinary laboratories for the size of sample used.

In verifying in the laboratory the method above suggested the author finds the following accuracy:

1. From the correct amount of cement to 6.0% less than the correct amount.
2. From the correct amount of sand to 4.5% more than the correct amount.
3. From 1.5% less than the correct amount of gravel to 3.0% more than the correct amount.
4. From 5.2% less than the correct amount of water to 51.0% less than the correct amount.

In view of the variation found in performing tests very similar to those of Mr. Dunagan, the author wishes to point out that Mr. Dunagan may be a little over-confident when he ventures to establish the water-cement ratio by use of his test.

Readers are referred to the JOURNAL *for June 1930, to which this discussion is continued.*

Shrinkage Measurements of Concrete Masonry*

BY W. D. M. ALLAN†

Mr. Allan's paper clearly indicates that concrete masonry units air cured after desired strength is obtained by moist curing, show less movement, pointing the way to reduction or elimination of cracking in concrete masonry walls of considerable area, caused by shrinkage of units laid up in a saturated or nearly saturated condition.—Editor.

INTRODUCTION

In recent years there has been a rapidly increasing use of concrete masonry in larger buildings resulting in large wall areas of above grade construction. With these extended wall areas, the occurrence of extensive shrinkage after the application of the finish coat of stucco or plaster may cause the formation of unsightly plaster cracks.

While the occurrence of plaster cracks in concrete masonry has been due mostly to faulty construction such as inadequate footings or improper bonding between different types of masonry, there are instances where such cracking was caused by wall shrinkage. Preliminary studies have indicated that moisture content of the units at the time the wall is erected is an important factor affecting the shrinkage of the hardened concrete masonry wall.

PURPOSE AND SCOPE

The purpose of this investigation was to determine the rate and extent of shrinkage occurring in concrete masonry panels as affected by moisture content of the unit at the time the wall is laid. This report gives the preliminary results of these studies and has for its purpose the development of information on which

*Presented at 26th Annual Convention, Feb. 1930.

†Manager, Cement Products Bureau, Portland Cement Assn., Chicago.

to base recommendations which will aid in reducing or eliminating the undesirable shrinkage.

It is realized that this investigation will not determine to what extent a wall may shrink without causing plaster cracking, as this is also a function of the kind of plaster used, but will point the way to reducing the possibility of their occurrence, due to the shrinkage of the concrete masonry wall.

This study includes the measurement of shrinkage on 24 masonry panels, and compression and absorption tests on 72 (8 x 8 x 16-in.) concrete masonry units. The units which were made with three different aggregates, (sand-gravel, haydite and cinders) were erected in panels and tested for strength and absorption at the age of 2, 7, 28 and 90 days.

MANUFACTURE OF BLOCK

The 360 three-oval-core 8 x 8 x 16-in. concrete units used in this investigation were made in the laboratory of the Portland Cement Association, using an Anchor Junior Stripper Block machine. The concrete was mixed in a 3-cu. ft. Blystone paddle mixer using 0.40 sack batches producing from 6 to 8 block per batch. The materials for the individual batches were weighed separately and added to the mixer. Each batch was mixed one minute dry and five minutes wet. Water was added to produce the desired consistency, as indicated by the appearance of water web markings. The block were tamped twelve times, four tamps in each of three layers. Immediately after stripping, all block were moist-cured 48 hr. in a room where the temperature was maintained at 70° F. and the relative humidity was about 100%. The block were then stored in the laboratory in such a manner that air circulated around each individual unit until laid up in the wall panel at ages of 2, 7, 28 and 90 days after molding. In addition to the block used in the construction of the test panels, block were made for strength and absorption tests and to determine the rate of loss of moisture during the period of air-curing.

Data of the different mixes are shown in Table 1:

SHRINKAGE MEASUREMENTS

The panels for shrinkage measurements were laid up as shown in Figs. 1 and 2 using a 1:1:6 portland cement-hydrated lime

TABLE 1

Aggregate	Mix by Vol.	Block per Sack	Compressive Strength Lb. per Sq. In. Gross Area				Moisture Content % by Weight			
			2d.	7d.	28d.	90d.	2d.	7d.	28d.	90d.
Elgin Sand-Gravel	1–9	22	780	960	1400	1480	5.3	3.2	1.1	0.8
Haydite	1–5–5 (Damp loose)	20	460	700	990	1060	19.3	7.3	4.9	2.8
Cinders	1–7	17	240	440	690	770	16.6	6.6	3.1	3.1

mortar with mortar joints ⅜-in. thick, the mortar covering the face shells and the end webs. The panels were built nine units high. The shrinkage was measured over eight units and seven mortar joints by means of gage plates placed on the floor and on top of the eighth unit. The gage plates were placed about 2 in. in from each end of the one unit wide panel. The bottom plates were equipped with two stainless steel plugs placed twelve inches on center and the top plates with adjustable round pointed stainless steel screws which were fastened in position by means of thumb screws. Four sets of gage lines approximately 64.6 in. long were used for each panel. The gage lines were set while panels were being erected. Initial readings to determine the original length of the panel were made within ½ hr. after erection in all cases. Duplicate panels were made for each condition studied and the shrinkage plotted in Figs. 3 to 7 is based on the average of readings on eight gage lines, 4 each on two separate panels. Readings were made at daily intervals for the first 30 days after erection and less frequently thereafter. The room in which the panels were erected was on the first floor of the building. Temperature readings made when the panels were measured ranged from 69° and 82° F. Relative humidity readings made at the same time as the temperature readings showed a maximum of 77% and a minimum of 32% with an average relative humidity of 53%. The air conditions of the room were such as to simulate the exposure conditions of interior wall construction.

The measurements reported in this progress report continued from May through December, 1929. These include readings taken over a period of one to three months when the building was

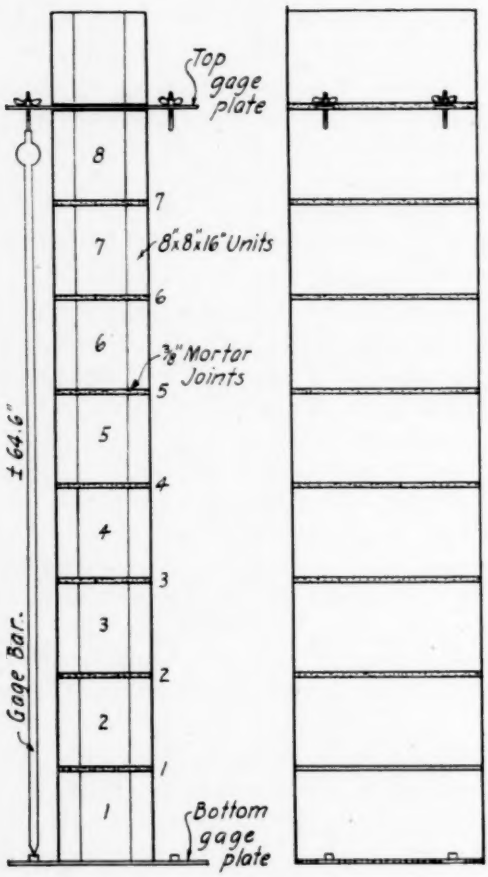

FIG. 1—PANEL DETAIL AND GAGE BAR

steam heated for all panels. Measurements were made with an Ames .001 inch dial mounted on a 5/8-in. round steel rod of proper length, which was pointed on one end. The length of the rod was

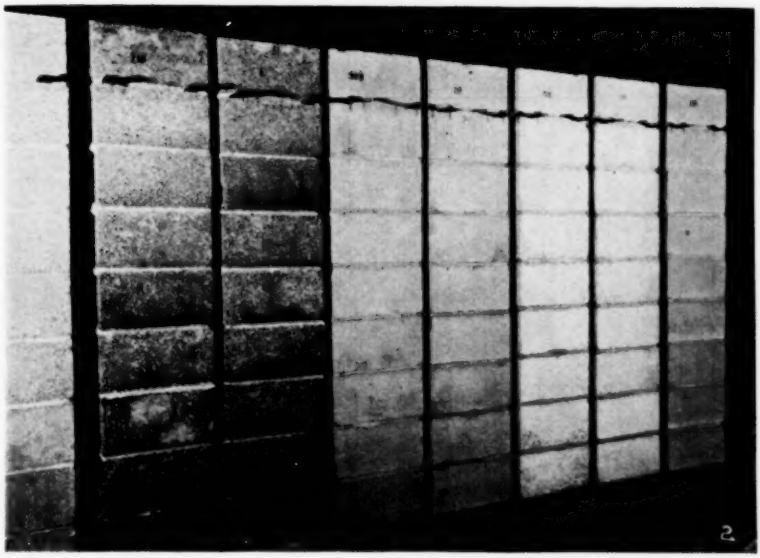

FIG. 2—PANELS ERECTED FOR MEASUREMENT

checked daily by means of a standard bar composed of a 2 by 2-in. steel channel fitted with a similar gage points as those used on the panel. Corrections for temperature as affecting the gage length were not made due to the fact that the standard bar readings compensated for any differences in readings due to this factor. Shrinkage of the 1:1:6 mortar was considered to be constant for all panels.

Effect of moisture content of units when laid on subsequent shrinkage. The panels for these measurements were erected with: 2-day old units, saturated; 7-day old units cured 5 days in the air of the laboratory; 28-day old units air-cured 26 days; and 90-day old units air-cured 88 days. Measurements were made on two panels for each age for each of the three aggregates studied —sand-gravel, haydite and cinders. The principal data and the more important relationships are plotted in Figs. 3 to 7.

Shrinkage of sand-gravel panels. Fig. 3 shows shrinkage curves obtained from plotting the measurements of the sand-gravel panels for each of the four conditions studied.

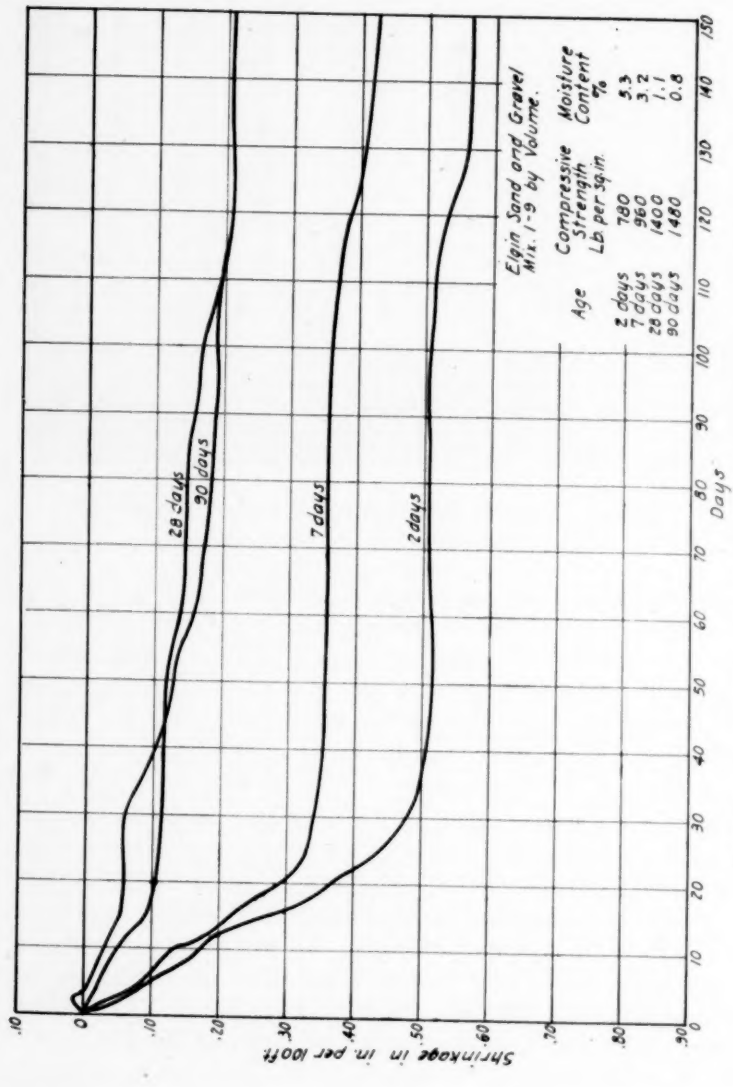

FIG. 3—SHRINKAGE OF SAND CONCRETE MASONRY PANELS

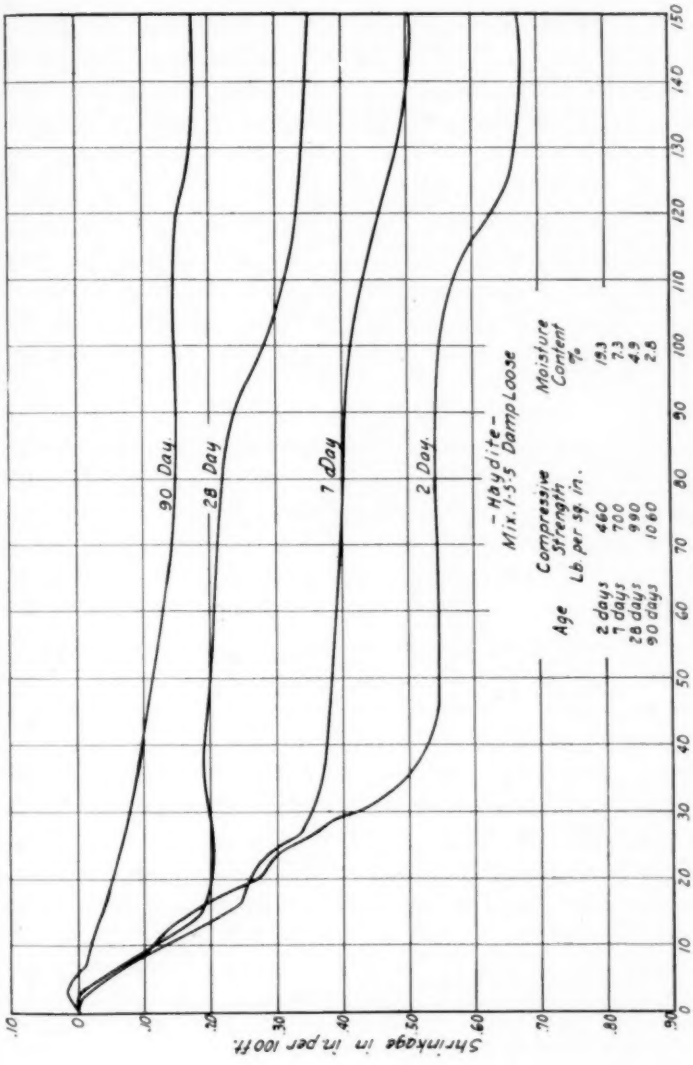

FIG. 4.—SHRINKAGE OF HAYDITE CONCRETE MASONRY PANELS

The panels erected with *saturated two-day old units* containing 5.3% moisture contracted at a fairly uniform rate for the first 30 days reaching a stationary condition in about 48 days. The total shrinkage at that time was equivalent to about .51 in. per hundred feet of wall. This length remained unchanged until the panel was about 115 days old when the building was heated which caused a further shrinkage of approximately .05 in. in a period of 15 days making a total shrinkage at 150 days of .56 in. per 100 ft.

The panels erected with *seven-day old units* (air cured 5 days) contained 3.2% moisture at the time of erection. These panels contracted at a uniform rate for about 25 days with a stationary condition being reached in about 40 days with a shrinkage of about .35 in. per 100 ft. Further shrinkage similar to that obtained with the two-day panels occurred when the building was heated. Total shrinkage 150 days after erection was .42 in. per 100 ft.

Panels erected with *28-day old units* (air-cured 26 days) containing 1.1% moisture contracted .10 in. in the first 15 days. At 90 days the total contraction was .15 in. Further shrinkage at about the same rate and extent occurred when the building was heated, making a total contraction at 150 days of about .2 in. per 100 ft.

The panel erected with *90-day old units* contracted at a slightly slower rate for the first 15 days than the 28-day panel. Then further movement ceased until the heating of the building occurred when the panels were about thirty days old, causing a further contraction equal to .15 in. per 100 ft. at 60 days. After this period the length of the panel decreased slowly to .20 in. in 100 ft. at 150 days.

A study of these curves indicates that the air curing of sand-gravel units for 5 days reduced the shrinkage occurring in the first 150 days to about 75% of that obtained with saturated units. Similarly, 26-day air curing and 88-day air curing reduced the shrinkage to about 35% of that obtained with the saturated units.

Shrinkage of haydite masonry panels. Fig. 4 shows the shrinkage curves obtained from plotting the measurements of the Haydite panels for each of the four conditions studied.

The panels erected with *saturated two-day old units* contained 19.3% moisture when laid up and contracted at a uniform rate for the first 45 days with a total shrinkage of about .54 in. per 100 ft. of wall. This length remained unchanged until the panels were about 105 days old when the building was heated which caused a further shrinkage of approximately .10 in. in a period of about 20 days, making a total shrinkage at 150 days of about .67 in. per 100 ft.

Seven-day old units (air cured 5 days) when laid up in a panel contained 7.3% moisture, contracted at a uniform rate for about 30 days and had a total contraction during this period of about .35 in. They contracted slowly until 100 days old when further contraction was noted due to the heating of the building. Total shrinkage was equivalent to about .50 in. per 100 ft. at 150 days.

The units for the *28-day old* panels (air cured 26 days) when erected contained 4.9% moisture and contracted at a fairly uniform rate for 15 days with a total shrinkage of about .19 in. per 100 ft. No further shrinkage occurred until the panel was about 85 days old when it contracted another .10 in. due to the heating of the building. Total shrinkage for 150 days was .35 in. per 100 ft.

The panels erected with *90-day-old* units containing 2.8% moisture expanded slightly during the first two days when contraction started and continued at a fairly uniform rate for about 75 days. This long period of shrinkage was probably due to the influence of heating the building which occurred 15 days after erection. The total shrinkage during this period was .15 in. per 100 ft. The length of the wall remained practically unchanged for the remaining period plotted, the total shrinkage being about .17 in. per 100 ft. for 150 days.

A study of the curves in Fig. 4 indicates that the air curing of the Haydite units for five days reduced the shrinkage for the 150 day period to about 75% of that obtained with saturated units. Similarly 26 days of air curing reduced the shrinkage to about half that obtained with the 2-day old saturated panel. Eighty-eight days of air curing reduced the shrinkage to about 26% as much as the 2-day panel.

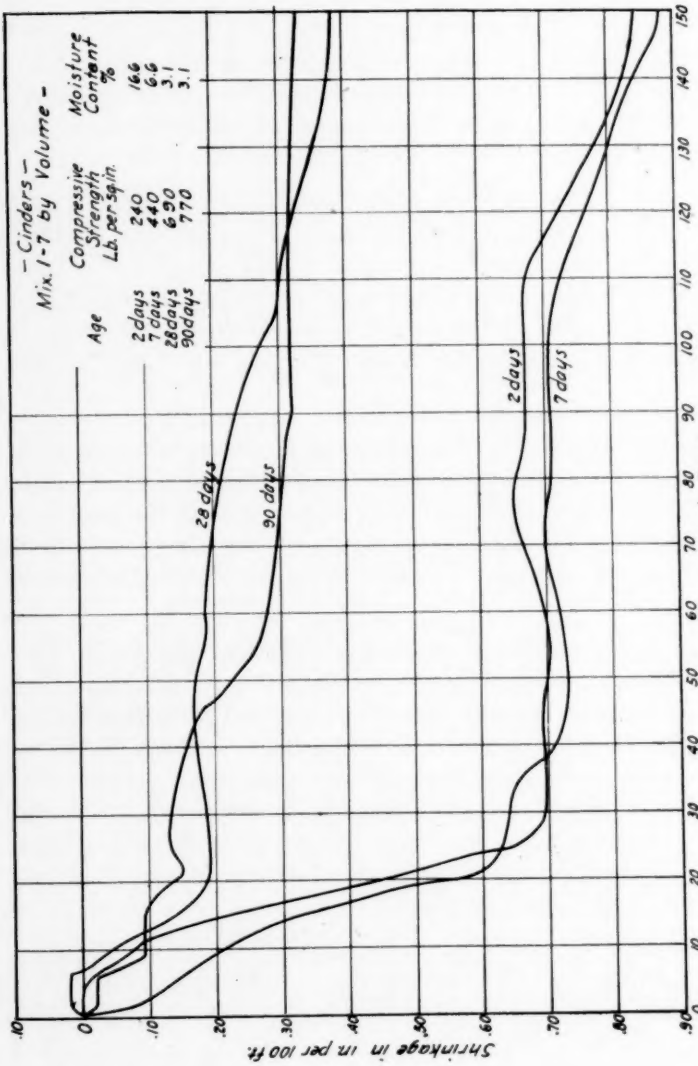

FIG. 5—SHRINKAGE OF CINDER CONCRETE MASONRY PANELS

Shrinkage of cinder concrete masonry panels. Fig. 5 shows shrinkage curves obtained from plotting the measurements of the cinder masonry panels for each of the four conditions studied.

Panels erected with *saturated two-day old* units containing 16.6% moisture contracted at a uniform rate for about 28 days at which time the total contraction was .70 in. per 100 ft.

The panels then remained stationary for a period of about 90 days when further shrinkage occurred due to the heating of the building. This proceeded at a uniform rate for about 25 days and the total shrinkage at 150 days was about .95 in. per 100 ft.

Shrinkage started immediately after erection of this panel built with *seven-day old* units containing 6.6% moisture. They contracted at a fairly uniform rate for 25 days with a total contraction of about .63 in. per 100 ft. Shrinkage continued at a slower rate for the next 15 days with no further change until the building was heated. Then shrinkage proceeded slowly until the total contraction was about .98 in. per 100 ft. at 150 days.

The panels built with *28-day-old* units containing 3.1% moisture expanded slightly during the first five days after erection. Shrinkage then set in which continued at a uniform rate until the panel was about 20 days old; the total shrinkage at this time being about .19 in. per 100 ft. From this point onward the length remained stationary until the panel was 85 days old when further shrinkage occurred due to the heating of the building and continued until at 150 days the total shrinkage was about .39 in. per 100 ft.

Units for the panels erected at *90 days* contained the same percentage of moisture as the 28-day-old panels. They contracted slightly during the first six days when the rate increased rapidly until the panel was 10 days old. having a total contraction of about .10 in. per 100 ft. Shrinkage then continued at a fairly uniform rate until the panel was about 35 days old when the rate was increased due to the heating of the building. A period of equilibrium was reached when the panel was about 90 days old and had contracted about .32 in. per 100 ft. The total contraction at the end of 150 days remained .32 in. per 100 ft.

A study of the shrinkage curves obtained with the cinder masonry panels indicates that a five-day air curing had little, if

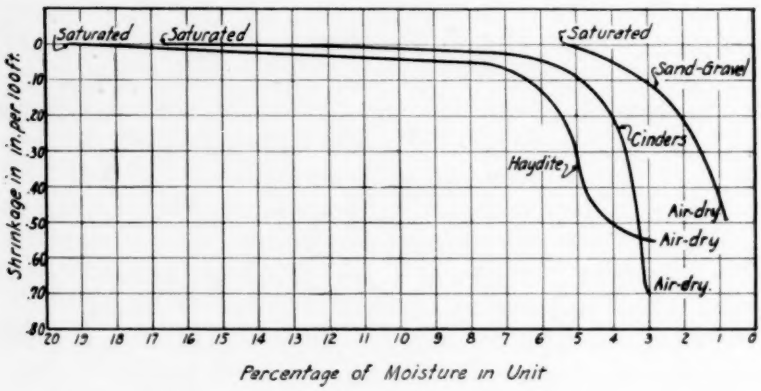

FIG. 6—RELATION OF MOISTURE CONTENT TO SHRINKAGE

any, effect upon the reduction of shrinkage of cinder concrete masonry. Both the two-day and seven-day panels shrank about the same extent which was equivalent to about .95 in. per 100 ft. The shrinkage of the 28-day and 90-day panels were fairly similar throughout the period and was about 37% of the shrinkage which occurred with the 2-day and 7-day panels.

In Fig. 6 is shown the relation of moisture content of the unit in the panel, as measured by the moisture content of separate units stored in the same condition, to the shrinkage occurring in the panel. The values plotted are for the panels erected with units (saturated) at the age of two days for each of the three aggregates studied.

Saturated units in the sand-gravel panels contained 5.3% moisture at the time of erection. The shrinkage of the panel followed closely the loss of moisture in the units. The rate increased rapidly as the air dry condition of the unit was approached. After the air-dry condition was reached, further shrinkage practically ceased until the building was heated.

For the haydite panel, similar studies show that the reduction in moisture content of 12% in five days by evaporation was accompanied by a shrinkage in the panel of only .05 of an inch per hundred feet. From this period on, however, the shrinkage fol-

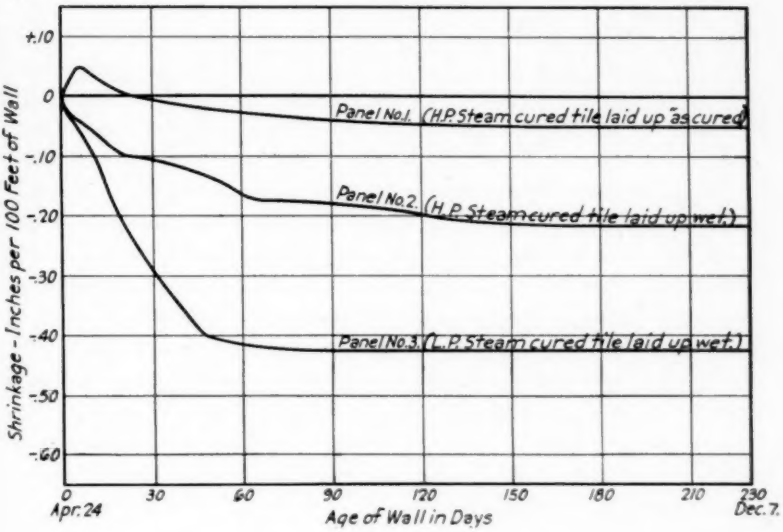

FIG. 7—EFFECT OF HIGH PRESSURE STEAM CURING ON SHRINKAGE OF CONCRETE MASONRY PANELS

lowed closely the loss of moisture in the units until a room-dry condition was reached. The reason for the small amount of shrinkage caused by the loss of 12% of moisture from the saturated condition may be explained by the porous nature of the aggregate, the water being mechanically held in the aggregate and in excess to the absorption of the material in the unit. The cinder panel showed similar action to that of the haydite panel, shrinking only .03 of an inch with a loss of moisture content of 10% in five days. After this period the shrinkage increased rapidly as the units approached a room-dry condition and practically ceased after this condition was reached.

A study of the three curves shows the influence of the moisture content of the unit on the shrinkage and emphasizes the necessity of having the units as near an air-dry condition as possible before they are laid in the wall. The loss of the last 2 or 3% of moisture was accompanied by the major portion of shrinkage occurring in the panels.

Effect of different methods of curing. In addition to the measurements made on the moist-cured specimens interesting comparisons can be made of the effect of high pressure steam curing and moist-curing on the amount of shrinkage which will occur in the masonry wall. Shrinkage studies reported in a paper on "Some Tests of Concrete Masonry Units Cured with High Pressure Steam"* which were made concurrently with the shrinkage studies reported in this paper show the following effects on high pressure steam curing on the shrinkage of a masonry wall. The three curves as plotted in Fig. 7 show that the maximum change for Panel No. 1 which was high pressure steam-cured for 12 hours at 100 lb. steam pressure contracted .05 in. per 100 ft. for a period of 230 days. Panel No. 2 was erected with similarly cured units which were saturated just prior to erection and contracted approximately .22 in. per 100 ft., while the low pressure steam cured units cured in the usual manner in the steam tunnel contracted about .45 in. per 100 ft. of wall.

When comparisons were made on the measurements of these three panels with those of Fig. 3 they clearly indicated that high pressure steam curing will greatly minimize the shrinkage occurring in a concrete masonry wall when laid in a moist condition.

While these shrinkage studies were made on panels containing 3 different aggregates, direct comparisons are not made because of the differences existing in the grading, moisture content at time of erection and cement content of the units made with these aggregates.

SUMMARY

The most outstanding indications from these studies to date are the following:

(1) That the use of air-dry concrete masonry units in wall construction will reduce the shrinkage to 20-35% of that occurring when saturated units are used.

(2) That the use of units which are only partially dry, containing 2 to 5% of moisture by weight, will result in the major amount of shrinkage which will occur in a concrete masonry wall.

*P. M. Woodworth, *Proceedings American Concrete Institute*, Vol. 26, p. 504, (JOURNAL, Feb., 1930).

(3) For the conditions of air curing used in these studies, 26 days of air curing was generally sufficient to obtain blocks that would result in the minimum amount of shrinkage when laid in the wall.

RECOMMENDATIONS

As the results of these tests indicate that the use of air dry units will practically eliminate wall cracking due to shrinkage of the concrete masonry unit, it is recommended that the products manufacturer should, after the curing period which is necessary to develop the desired strength of the concrete, stock pile the units in such a manner that the air can circulate freely through them, preferably under cover. Precautions should also be taken to keep the units dry after they are delivered on the job.

CONVENTION DISCUSSION

C. M. Chapman, (New York): Do the data from these tests differentiate the shrinkage of the mortar joint from the shrinkage of the block itself? If separate measurements were made of the block, then perhaps by a little mathematics, we might determine the amount of shrinkage that took place in the mortar joint. There is undoubtedly a difference between the shrinkage one might get from masonry units, say block size as compared with masonry unit of brick size. In the latter case the shrinkage of the mortar joint becomes many times more important. The re-expansion due to absorption from the mortar joint which is shown in some of those curves after ninety days, is in part due to the mortar giving up its moisture to the block. In the case of brick that would be equivalent to quite a high percentage of moisture, and consequently a greater relative expansion before shrinkage started in if the brick were fairly dry at time of laying. I have found several cases of rather serious trouble due to volume change of concrete, sand-lime, and ordinary clay brick in walls, and they were in chimneys where we do not like to have cracks occur. It would be very helpful if this same sort of work could be carried on for concrete brick masonry walls. I would suggest also that inasmuch as this investigation was confined to concrete block, that the title indicate, for the sake of accuracy, that it does not include all concrete masonry units.

Raymond E. Davis, (Berkeley, Cal.): I have listened to this paper with a good deal of interest inasmuch as I have, for the last several years, in one way and another, been connected with investigations having to do with volume changes in mortars and concretes. I am struck with the idea that, while knowing the shrinkage is a very desirable thing, we must know, for these particular units, something of the total volume change that they undergo between the two extremes; that is, not only will concrete, on drying out, shrink, but upon absorbing moisture, will also swell. And it would seem necessary, inasmuch as the block and concrete brick and all exterior construction, are subjected to weather conditions, that we should know something about the total volume change we may expect each one to undergo. So far as my observations extend, the aggregate itself seems to have a rather important bearing upon what the total volume change may be, even among our stone aggregates, being materially less, we will say, for limestone than for sandstone. I have in mind right now an incident that came to my attention in the last two months, when I was called in to examine a warehouse where the concrete was cracking very badly and some of the columns seemed to indicate that they were in tension instead of being in compression. The warehouse was a one story structure with columns at intervals and spandel beams across the top supporting rather light roof loads, and in between the beams on the columns were blocks, and I examined the columns and found near the top of the junction was a spandel beam. In each corner I found I could look in and see the steel, which seemed peculiar for a column inasmuch as we expect it will be a compression member. Upon further investigation, it appeared perfectly certain that what had happened was that the block in this panel had expanded upon exposure to the rain, its volume had increased to the point where it had actually stretched those columns through its pushing up against the spandel beam, stretched them to the point where the steel was in tension instead of compression, and had actually gone beyond the yield point. I cite that to emphasize what seems to me to be the need in units of this kind for providing for expansion and contraction by some sort of a plastic or yielding joint in a good many instances. With regard to what Mr. Chapman has said, there is also a great difference depending upon the

mortar used in the joints, and I refer now particularly to brick walls. A brick wall or a brick pier, immediately after it is laid, instead of shrinking expands for a considerable period of time. That is true regardless of the mortar, whether it is a lean mortar or a rich mortar, cement, lime mortar or whether it contains some other admixture, but rich mortar expands more than lean mortar, and in those mortars containing a high magnesium lime, the expansion is much more than for those containing a high calcium lime. These continue to expand for a considerable time. Walls that have been under observation indicate that even in dry air with constant, fifty per cent relative humidity, which is fairly low, expansion continues for a year or more before they begin to shrink in dry air, but only in rare instances does the volume of that brick masonry become appreciably less than on the day it was laid, and when the rains come along and beat upon the house, the water runs into the joints and you get a considerable expansion. Just the day before I left, I was called into a conference in San Francisco to examine a twenty-five story building faced with terra cotta, in which there were possibly ten thousand cracks in the terra cotta due to the increase in the volume of masonry behind that terra cotta facing, due to the seepage of the water in through to the mortar joints, and it brings out again the importance of knowing something not only of what shrinkage occurs in the unit, but will occur with expansion upon the absorption of moisture.

P. H. Bates, (Washington): I remember in 1917 attending a meeting in this hotel of a group that was very much interested in the exterior of this hotel at that time. You will notice, as you come into the hotel, that one entrance, in fact the entire face of the sides of this hotel, is terra cotta, and if you have noticed particularly, you will notice that the terra cotta is pretty badly cracked. The meeting I attended in 1917 was the meeting of the National Terra Cotta Society, and the building was as badly cracked then as it is now, and of course, having the evidence right at hand, it was a matter of considerable discussion. At that time there were practically none of the so-called plastic mortars on the market. By plastic mortar I mean something entirely different from portland cement or portland cement lime mortar—a

material largely organic, which is supposed to maintain its plasticity over a period of years. That was one item much discussed at that time, the possibility of developing a plastic mortar to be used at certain levels in the building by which the settlement stresses or this differential volume expansion of the back and front of the walls might be brought into equation. That has, in the case of a number of materials, become quite customary. For instance, if a terra cotta manufacturer can persuade the architect, and he usually can, there are put in at various levels in terra cotta veneered buildings, a complete course of plastic mortar, and I think we will have to come to that in other types of masonry. Mr. Davis also called attention to the brick walls laid up with cement mortar. We have noticed this difference between mortars having different types of lime as a plasticiser. We are going to come, in due course, to look upon limes and not simply lime, as we are coming to look upon cements and not simply cement, for they have different properties; in masonry construction to obtain volume constantly, is a matter of extreme importance.

Readers are referred to the JOURNAL *for October, 1930, for discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1930.*

ABSTRACTS

of the current literature of cement and concrete

PREPARED UNDER THE DIRECTION OF THE
AMERICAN CONCRETE INSTITUTE
COMMITTEE ON ABSTRACTS

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MATERIALS

AGGREGATES

Some additional factors in the prediction of tensile strength of sand mortars. H. WALTER LEAVITT, JOHN W. GOWEN AND WESTON S. EVANS. *Maine Technology Exp. Sta. Bull.* No. 24, *Sand Study* No. 5, Jan. 1930, V. 16, No. 1, 12 pages.—Any accurate method of arriving at 28 day mortar strength of sands by means of short time tests would meet with almost instant approval in concrete field. This article continues the subject of Bulletin No. 21 by adding two additional factors—per cent of mixing water and colorimetric test for organic impurities—to mechanical analysis data. All factors used in this prediction equation can easily be obtained within a few hours after receipt of sand sample. A more accurate method than A. S. T. M. field method for determining organic impurities of sands was tried and found to have less value than "Field Method." Therefore "Field Method" results were used in this study. By means of multiple correlation a prediction equation was derived: 28 day Tensile Strength = 149.27 + 3.97 (R-No. 8) + 1.52 (R-No. 14) — 1.29 (R-No. 28) — 5.24 (R-No. 100) + 0.24 (P-No. 100) + 24.25 (Per cent Water) — 73.73 (Color). "R-No.—" signifies per cent of sand retained on that size sieve. Results of application of this equation to 102 Maine sands are tabulated. Average error without regard to sign was 50.1 lb. per sq. in., and with regard to sign was —23.6 lb. per sq. in.

Conclusion of this study was: The mechanical analysis data, per cent of mixing water, and colorimetric test for organic impurities of sands give valuable information for prediction of the 28 day tensile strength of sands. Other important factors, however, must be measured to make more accurate predictions. (cf. The Relation of the Grading Characteristics of Maine Sands upon their Tensile Strength, Gowen, J. W., Leavitt, H. W., and Evans, W. S., pp 1-12, June 1928. *Maine Technology Exp. Sta. Bull.* No. 21. Also, Mechanical Analysis of Sand, Gowen and Leavitt, pp. 1-20, 1927. *Maine Technology Exp. Sta. Bull.* No. 18.)—H. W. LEAVITT

Specifications for mortars and concrete. "Anweisung fuer Moertel und Beton." 1928, 2nd ed. Wilhelm Ernst und Sohn, Berlin. R. M. 4.50. Reviewed in *Zement* (Germany), Jan. 1930, V. 19, No. 3, p. 64.—Book gives directions for correct valuation and choice of mortars. Special consideration is given to mixing of concrete, size of aggregates, water requirement and surface finish of concrete. Three main chapters deal with (1) raw materials for mortar and concrete, (2) mortar and concrete and (3) construction control. Valuable data for calculations are furnished in 15 tables.—A. E. BEITLICH

Novel sand-gravel transportation. *Rock Products*, Jan. 18, 1930, V. 33, No. 2, p. 25-29.—Unusual sand classification methods and aerial tramway transportation from river deposits to plant of Ross Island Sand and Gravel Co., Portland, Ore. Ready-mixed concrete plant of 500-yd. per day capacity also operated, aggregates being proportioned by weight. Shipments are made in "bath-tub" bodies on trucks. Seventeen illustrations.—J. J. LANDY

Synchronous motor drives for gyratory crushers. W. C. KING. *Rock Products*, Nov. 9, 1929, V. 32, No. 23, p. 104.—Comparison of relative power factors of synchronous and induction type motors for crusher drives. Illustrates typical examples of modern drive—a 7-ft. cone crusher by a 250-hp. synchronous motor.—J. J. LANDY

CEMENT

Alumina cement and portland cement strength properties, 1929. ALEXANDER HASCH. *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Jan. 1930, V. 82, No. 5-6, p. 40.—Comparison of strength properties of different kinds of cement examined during 1929 by laboratory for testing construction

materials of the city of Vienna.—A. E. BEITLICH

New wet-process cement plants in the United States of America. H. ENGEL. *Zement* (Germany), Jan. 1930, V. 19, No. 4-5, p. 72-8, 96-102.—Description of 14 new American wet-process cement plants. Special consideration is given to storage and transportation of raw and finished materials, mixing of slurry, kilns and coolers, waste heat economy and dust collection.—A. E. BEITLICH

The influence of additions to cement raw materials on fineness in the wet grinding process. P. P. BUDNIKOFF, G. W. KUKOLEW AND W. M. LESCHOJEW. *Zement* (Germany), Jan. 1930, V. 19, No. 5, p. 96.—Experiments were made with a raw mix consisting of marl and chalk. Both materials were crushed in jaw-crusher and sieved through two sieves with 1 and 4 meshes per $cm.^2$ Residues on 4-mesh sieve were mixed together in desired proportions. Two porcelain ball mills with same number and weight of balls were fed each with 1 kg. of raw mix. One mill was filled with ordinary water and the other one with a $0.04/N\text{-}Na_2CO_3$ solution and a 0.1 per cent molasses solution. The grinding time was 30 min. The sieve analysis showed much better results in the case of grinding with these solutions.—A. E. BEITLICH

Safety devices and possibilities of increasing safety in cement plants. PROESCHEL. *Zement* (Germany), Jan. 1930, V. 19, No. 4, p. 87-8.—New safety devices in modern cement plant are described. Improvements are shown in raw mill, dryers, raw mix bins, burner building, coal mill and dust collectors.—A. E. BEITLICH

The pneumatic air separation. A. B. HELBIG. *Zement* (Germany), Jan. 1930, V. 19, No. 2, p. 25-8.—Applications and advantages of different types of air separators in grinding process of cement industry are described. Formula to calculate efficiency is given.—A. E. BEITLICH

The recent development of the Japanese portland cement industry. MITSOZU FUJII. *Zement* (Germany), Jan. 1930, V. 19, No. 4 and 5, p. 89-92, 113-5.—A report on the developments in modern cement plants. Raw materials and their deposits are described. Data are furnished about location of plants, their output, machineries, kilns, dryers, grinding equipment and power consumption. Standard specifications are characterized and description given of the economical development, cement consumption and cement organizations.—A. E. BEITLICH

Structure and chemical resistance of Concrete. (See PROPERTIES OF CONCRETE.)

Slide rule for calculating the compounds in portland cement. L. A. DAHL. *Rock Products*, Nov. 9, 1929, V. 32, No. 23, p. 50.—Dahl's slide-rule designed to give usual portland cement compounds from clinker analysis. Each compound calculated independently. Special scales printed to size to fit ordinary Mannheim rule. Mathmetical basis of design and typical examples of operation given.—J. J. LANDY

The calculation of the power consumption of tube mills. H. DREYER. *Zement* (Germany), Dec. 1929, V. 18, No. 50, p. 1434-9.—Previous tests by Fischer had indicated that the correct number of r. p. m. for tube mills depends on the diameter of the tube. The process in the interior is described in the following manner. The balls and the charge are raised by friction, then the balls fall back again and crush the material. Each ball a in the distance ϱ from the tube axis obtains the velocity: $v = 2\,\varrho \times \pi \times \frac{n}{60}$. When the partial force of the gravity, which is directed towards the axis, equals the centrifugal force, influence of motive power of tube mill upon the ball ceases and the latter moves back to its starting point in the curve of a parabola. The question is to determine amount of work which is necessary to accelerate and lift balls and charge. In order to calculate this one can imagine the entire mass of the filling being concentrated in the particle a. The amount of work A for one revolution is then:

$A = \frac{1}{2}m \times v_g^2 + 4m \times v_g^2 \times \cos^2 \alpha$. ($m$ is the mass and α is the angle between the point in which the influence of the motive power upon the particle a ceases and the horizontal line through the center of the tube mill.) The amount of work per second can be calculated from diameter of the tube mill and distance between inner surface of layer of charge and center of tube. The average r. p. m. of particles is z, the best degree of the filling is 0.4, that means that 0.4 of the total volume of the mill is filled by the balls and charge. Q is the weight of both in kg. and D is the diameter of the tube mill in meters. $N = 6.9 \times \frac{Q}{1000} \sqrt{D}$ in H. P. Further calculations take into account the pressure upon the bearings by the weight of the tube mill, balls and charge and the bouncing against the walls. The following formula was derived: $N = C \times \frac{Q}{1000} \sqrt{D}$ in H. P. When the charge consists of cement the value for C is 9.5 for flint pebbles, 8.5 for big steel balls and 8.2 for small steel balls. Examples of the practice furnish proof of the accuracy of these formulas. Another calculation deals with power consumption for starting of tube mill. An increase in speed does not increase, but decreases efficiency of mill and leads to destruction because balls drop upon unprotected wall and not upon charge.—A. E. BEITLICH

Chemistry and physics of the combustion of gaseous fuels. G. E. SEIL, H. A. HEILIGMAN, and C. N. WITHROW. *Ind. Eng. Chem.*, 1930, V. 22, No. 2, p. 179.—Reasons for desiring to control chemical constituents of furnace atmosphere are discussed, and chemical and physical effects of excess carbon monoxide, excess oxygen, neutral atmosphere, and varying atmosphere are enumerated. Atmosphere in gas-fired tunnel and periodic kilns is traced throughout burn and compared with coal-fired kilns. Control of kiln atmosphere when using solid, liquid and gaseous fuels is discussed.—(From Portland Cement Ass'n. Literature Supplement.)

The rotary kiln in cement manufacture. W. GILBERT, *Cement and Cement Mfg.* (England), 1930, V. 3, No. 1, p. 41.—A discussion of operation of rotary kiln from point of view of output and fuel economy, and an attempt to elucidate laws which govern rate of heat transmission from hot gases to raw materials in kiln, and from hot clinker to air passing through cooler.—(From Portland Cement Ass'n Literature Supplement.)

The testing of cement. G. HAEGERMANN. *Cement and Cement Mfg.* (England), 1930, V. 3, No. 1, p. 69.—Essential properties of cement are: soundness, setting time and strength; fineness, volume and specific gravity are of less importance. Accelerated test for soundness is more universally used but experiments have proven that cements not complying with accelerated test may be sound in practice. Accelerated tests do not indicate expansion due to presence of gypsum and should be supplemented by observation by pats placed in cold water. Initial set indicates beginning of setting reaction but final set represents an arbitrary degree of hardening of no special importance and should be replaced by strength specification at an early age. Tensile and compressive tests indicate different qualities of the cement. Since tensile stresses are seldom met with in concrete structures, the true strength of mortars is best judged by its compressive strength.—(From Portland Cement Ass'n Literature Supplement.)

On the testing of cements. HANS KUHL. *Cement and Cement Mfg.* (England) 1930, V. 3, No. 1, p. 31.—Discussion of methods of testing portland cement, and evolution of cement specifications, emphasizing need for separate standard specifications for each building material. Specifications for specific gravity and weight per liter are considered obsolete. In future specifications will limit proportion of most finely ground particles rather than maximum residue on sieves of coarse mesh. Cements which meet strength specifications are certain to be ground fine enough, while cements ground too fine show decreasing strength when stored, and are more sensitive to increased water content than are more

coarsely ground cements. Tensile strength tests with plastic mortars are the best indication of utility of product. Preparation of test specimens should be mechanized so far as possible to insure uniformity of results. Work of Gensbaur and Prussing indicate relation between compressive strength, tensile strength and soundness which if confirmed by further research may lead to new specifications for strength and soundness of cements. This relation is also indicated by changes in the lime content of raw mix. Increasing lime content up to certain value increases both tensile and compressive strength, when this lime value reaches certain limit compressive strength still increases but tensile strength decreases. Further increase of the lime content results in decreased compressive strength and apparent unsoundness.—(From Portland Cement Ass'n Literature Supplement.)

Cement. T. MERRIMAN. *Paper No. 308*, In. Eng. Congress, (Japan) 1929, 16 pages.—Attention is directed to many differences found between different brands or samples of standard portland cements, attributed to chance combinations in the kiln. Argument is developed that law of average forbids formation and existence of a certain few specific compounds in cement clinker. In place of specific compounds the author proposes four type forms of combinations, of indefinite composition, to exist in cement and to determine its properties as follows: (1) Intimate combination of constituents; makes durable and resistant concrete of high strength. (2) Indifferent combination of constituents; makes strong concrete which tends to disintegrate. (3) Loose combination of constituents; makes concrete of good strength which disintegrates easily. (4) Poor combination of constituents; develops no strength in concrete and acts as an adulterant which readily disintegrates. Process of hydration is detailed the statement made that nature of hydration products changes with the amount of water used; hence the application of the cement-water ratio law. A hypothetical curve is presented showing rate of hydration plotted against alkalinity. This curve and accompanying argument are made to show zone of very slow rate of hydration at medium alkalinity and zones of rapid rate of hydration at both high and low alkalinities. Data are not experimental but are deduced from number of isolated tests and observations. Outstanding conclusion made in this part of the paper is that rate and heat of hydration are determined primarily by the alkali content of the clinker. To secure best cement, author suggests minimum of alumina, tensile strength at 28 days proportional to its lime silica index at given finess, no retrogression from 7 to 28 days, no serious retrogression on immersion in sodium sulfate solutions, and low solubility in sugar solution.—(From Portland Cement Ass'n Literature Supplement).

Essential properties of cement not yet guaranteed by the manufacture. OTTO GRAF. *Cement and Cement Mfg.* (England), 1930, V. 3, No. 1, p. 30.—Testing of materials for standard tests should be carried out so as to allow results of tests to be applied to actual practice. This requirement is not entirely met with cement. Compressive and tensile strengths are carried out on mortar specimens which are not true indication of results to be obtained with concrete. They give no indication of volume changes, resistance to corroding agents, workability or permeability.—(From Portland Cement Ass'n Literature Supplement.)

Electrical investigation of the setting and hardening of mixed cements containing iron blast-furnace slag. (See PROPERTIES OF CONCRETE).

The setting and hardening of cements. (See PROPERTIES OF CONCRETE).

Some physical properties of hydrated cements. (See PROPERTIES OF CONCRETE).

MISCELLANEOUS

The valuation of lubricants. ERNEST W. STEINITZ. *Zement* (Germany), Jan. 1930, V. 19, No. 2 and 3,

p. 38-40, 62-3.—Several kinds of lubricants, especially those used in cement industry, are characterized. It is shown that an extensive valuation is possible with simple means. Directions are given for determination of the spec. gravity, flash point, viscosity at different temperatures, asphalt contents of animal and vegetable oils.—A. E. BEITLICH

PROPERTIES OF CONCRETE

The resistance of reinforced concrete against sea water. H. E. SQUIRE. *Chimie Ind.*, 1930, V. 23, p. 92.—Sea water attacks reinforcement as well as concrete; latter is attacked in submerged portions and former above the water. It is necessary to prevent access of air and water into the mass to prevent corrosion. The structure should be very homogeneous. Good tamping in the forms is not sufficient to prevent porosity, which largely depends on the proportion of cement in the concrete. Quantity of water should be such as to not permit segregation. Too dry mixes are not recommended. Aggregate themselves should be resistant and non-absorbent. They should be graded. Washed pebbles are preferred; however, addition of fine sand or other finely divided material is favorable if added in well defined quantity. Concrete should be used as soon as possible after gauging. To augment resistance of concrete, waterproofing material, such as asphalt or hot China wood oil, may be used. Reinforcement may be coated with zinc or cadmium.—(From Portland Cement Ass'n Literature Supplement.)

Crazing and cracking on concrete surfaces. *Brit. Sci. Abs.* (England), 1929, V. 2, No. 11, p. 400.—Following precautions are suggested as means of preventing concrete surface crazing and cracking: (1) use of relatively lean (1:3 mixes), (2) reducing trowelling to minimum; (3) use of relatively coarse aggregates with low absorptions; (4) thorough and strictly controlled curing; (5) careful craftsmanship. — (From Portland Cement Ass'n Literature Supplement.)

The setting and hardening of cements. C. H. DESCH. *Cement and Cement Mfg.* (England), 1930, V. 3, No. 1, p. 73.—When water is added to cement the cement particles become coated with gelatinous layer in which small crystals subsequently appear. Hardness which results may be partly due to interlacing of crystals, but probably much more to drying of colloid mass. Products formed in hydration of cement must vary in composition according to proportions of reacting substances. Finer grinding of cement permits reactions to proceed more rapidly and more completely.—(From Portland Cement Ass'n Literature Supplement.)

Some physical properties of hydrated cements. R. E. STRADLING. *Cement and Cement Mfg.* (England), 1930, V. 3, No. 1, p. 19.—Discussion of volume changes occurring in hydrated cement includes curves showing the relative expansion which takes place when previously dried specimens of various building materials are immersed in water. With neat cement and cement mortars the expansion increases with increasing quantities of cement. With neat cement specimens of varying thickness from ⅛ to 2 in., rate and extent of expansion when stored in water, or contraction when stored in air, decreases as thickness of specimen increases. These changes in length result in formation of cracks in neat cement, but such cracks are less noticeable in mortars and concrete. In mortar these volume changes result in a concentration of stress around inert particles, these had to flow with consequent release of stress. Failing release by this means, minute cracks are produced in cement envelope around inert particles to allow the release of stress. If stress is sufficiently great major cracking may occur even in presence of inert aggregates.—(From Portland Cement Ass'n Literature Supplement.)

Electrical investigation of the setting and hardening of mixed cements containing iron blast-furnace slag. Y. SHIMIZU. *Paper*

No. 162, Research Inst. for Iron, Steel and Other Metals, Tohoku Imperial Univ., Sendai (Japan).—Examination of the electrical conductivity of mixtures of slag with portland cement or aluminous cement shows setting and hardening of slag is accelerated with increasing quantities of cements. Setting of slag corresponds with that of cement when portland cement is added in the proportion of 70 per cent to 30 per cent slag. Same is true of aluminous cement when present in proportion 60 per cent cement to 40 per cent slag. Compressive strength tests show increases in strength as cements are added to slag until above proportions are obtained, beyond which no further increases occur. Slag-aluminous cement (60 per cent aluminous cement plus 40 per cent slag, Patent No. 77913) is found to have strength curve very similar but slightly higher than straight aluminous cement, and decidedly higher than iron-portland cement.—(From Portland Cement Ass'n Literature Supplement.)

The adhesion of various cements to cast iron, mild steel and wrought iron. F. JASSOY. *Brit. Sci. Abs.* (England), 1929, V. 2, No. 11, p. 402-3.—Investigation was undertaken to determine whether and to what extent, adhesion between steel and cement in reinforced concrete can be attributed to formation of simple or complex iron silicates. Experiments were planned to examine the adhesion between portland, portland-blast furnace and aluminous cements and cast iron, mild steel or wrought iron (raw, planed and polished). Conclusion is that there is variation in strength of bond with time. Initially, aluminous cement gives higher bond strengths than portland blast furnace or portland cement. After 200 days, however, portland blast furnace cement gives considerably higher bond strengths than does either portland or aluminous. Type of metal has but little significance, although in these tests cast iron gave slightly higher bond strength. The polished surfaces gave slightly better results than the raw or planed surfaces. Bond is due to pure adhesion, not to chemical combination and is related to colloidal nature of cement. — (From Portland Cement Ass'n Literature Supplement.)

Structure and chemical resistance of concrete. RICHARD GRUEN. *Zeitschr. angew. Chem.* (Germany), 1929, V. 42, No. 46, p. 1070-4.—Resistance of concrete depends on its structure, chemical composition of the cement and chemical composition of the aggregate. Aggregates are in most cases very resistant. Cements can be divided into three classes, each one having a special place in the three-component system CaO-SiO_2-Al_2O_3: portland cement, slag cement and high alumina cement. Composition, constitution and hydraulic properties of these three cement classes are described. Equations given show hydration process of different constituents. Structure and strength of concrete depend on manner in which it is mixed and worked. Properties of dry, plastic and wet concrete are characterized. Influence of bases, acids and alkali waters upon hardening process and finished concrete is shown. Microphotographs of thin-sections and photographs of testpieces are given. The concrete can be protected against aggressive waters by protective coatings.—A. E. BEITLICH

Practical advices for concrete control. A. HUMMEL. *Zement* (Germany), Jan. 1930, V. 19, No. 3, p. 53-6.—Control of concrete generally consists of determination of only strength properties, which does not always prevent failure in construction. Several other examinations should be made on the job: control of mixture and water addition per m^3 of finished concrete, size of aggregates and sieve analysis. Empirical tables are only for rough calculations. Many factors depend on apparent density of cement and its necessary water requirement. Examples are given and short method for sieve analysis is described.—A. E. BEITLICH

The economical concrete mixture. "Die wirtschaftliche Betonismchung." KARL BRAUSEWETTER. (Germany), Wilhelm Ernst und Sohn, 1929. Reviewed in *Zement* (Germany), Dec. 1929, V. 18, No. 50, p. 1449.—Instructions for the testing of different

building materials, especially reinforced concrete, by means of test beams are given. Tests for setting time and soundness and the determination of the w/c-ratio for different aggregates are described.—A. E. BEITLICH

Bureau of standards investigations on cement, concrete, lime, gypsum and stone. *Rock Products*, Jan. 4, 1930, V. 33, No. 1, p. 137-141.—These progress reports cover: Cast stone, physical properties including absorption, resistance to freezing and thawing, etc.; reaction of water on high alumina cements; waterproofing agents for concrete; diatomaceous admixtures and methods of judging their qualities; constitution of portland cement, including the role of ferric oxide and magnesia; the cement reference laboratory; durability of bond between mortar and brick under freezing and thawing tests; water absorption of brick in an attempt to classify brick by absorption; compressive strength of brick walls, its relation to the transverse strength of the brick; compressive and transverse strength of brick strength of gypsum fiber concrete; expansion of gypsum fibered concrete during setting; soundness tests for lime; particle size distribution of hydrated lime; strength of commercial sand-lime brick; resistance of flooring materials; slate investigations—covering physical properties.—J. J. LANDY

Some additional factors in the prediction of tensile strength of sand mortars. (See MATERIALS.)

Specifications for mortars and concrete. (See MATERIALS.)

Alumina cement and portland cement strength properties. (See MATERIALS.)

The modulus of elasticity in arches in accordance with the parabolic stress variation. (See ENGINEERING DESIGN.)

Cement. (See MATERIALS.)

The testing of cement. (See MATERIALS.)

ENGINEERING DESIGN

BRIDGES

Overhead trusses carry arch centering for Pennsylvania R. R. Bridge at Philadelphia. WILLIAM H. FOWLER. *Eng. News Record*, Feb. 13, 1930, V. 104, No. 7, p. 282-284.—In construction of Pennsylvania Railroad's new concrete arch bridge over Schuylkill River in Philadelphia, problem of keeping river channel open for navigation was solved by use of overhead trusses for supporting arch centering. Structure consists of two five-centered concrete arches of 172-ft. 8 7/16-in. span and 31-ft. rise, each with heavily reinforced barrels 3 ft. thick at crown and 6 ft. 3¼ in. thick at springing line. Trusses for supporting centering are carried on pier and abutments on framed steel bents resting on I-beam grillages so designed that trusses may be shifted from one section of arch ring to next. Each truss is made up of twenty panels 8½ ft. each and two end panels of 11½ ft. each, having total span length of 193 ft. and rise of 30 ft. Four trusses are required, for west span and three for east span, trusses being spaced 8 ft. on centers. Adjustable tension rods 2 to 2¼ in. in diameter, arranged in pairs, support centering, of 6 in. wood lagging carried on pairs of 20.7- to 35-lb. 12-in. channels, curved to fit the required radius. Lagging for centering of each span is divided into seven panels for convenience in removal. Each panel is from 32 to 36 ft. wide and about 25 ft. long. Tension rods are carried on seats riveted to both sides of webs of 12-in. steel beam vertical posts of the trusses. Centering provided is sufficient for one ring section of each arch and will be used four times. Deflection of falsework under weight of concrete was computed and lagging set high enough so that arch ring after pouring would reach the desired elevation. For this adjustment, hanger rods were threaded at top and provided with nuts and lubricated bronze washers. To avoid excessive stresses in the hanger rods supporting the first pour, due to increments of truss deflection in the succeeding pours and under conse-

quent adjustments, rubber washers 6 in. in diameter and 1 in. thick were provided. The total deflection of the crown was about 3½ in. Article also gives details of the pouring and curing of the concrete and the removal of the centering.—D. E. LARSON

High concrete stresses used in design of viaduct of novel design. W. F. WAY. *Eng. News-Record*, Feb. 20, 1930, V. 104, No. 8, p. 327-329.—Eight 40-ft. spans are carried on tall columns without sway bracing. Girders for the two-column bents are 6 ft. wide. Deck slab has maximum depth of 17½ in. Joint Committee report of 1924 was used in design. A 1:2:3 mix, giving 3000 lb. per sq. in. concrete allowed stresses of 1200 lbs. per sq. in. in bending and 360 lbs. per sq. in. in shear. Loading assumed was 20-ton trucks, each with 32000 lbs. on the rear axle, passing at midspan with 100 per cent impact on the rear axle of one truck. Form work was simple and concrete was easily placed.—N. H. ROY

The modulus of elasticity in arches in accordance with the parabolic stress variation. RUDOLF KERN. *Beton u. Eisen*, Jan. 20, 1930.—Heft 2, V. 29, p. 29-34.—At second International Convention for Bridges and Buildings, in Vienna in 1928, desirability of an investigation of modulus of elasticity E for arches was pointed out. In case of concrete structures it is not sufficient to speak merely of E value without more specific explanation, since there are so many variables on which E depends. In order to show that it is not always correct to use the modulus of elasticity obtained from compressive tests, the case of an unreinforced concrete barrel arch of 40′ span and 10′ rise is considered. The concrete is assumed to have the stress-strain curve given by Morsch, "Der Eisen beton bau," 4th Ed., p. 43. A brief treatment is given of the rectangular concrete section acted upon by eccentric tensile or compressive forces. If a general equation is obtained, for the given concrete, in which the stress is expressed in terms of the second power of the strains, then the modulus of elasticity (tangent not secant) varies with the strains. In analysis this variation of E is considered by assuming a constant E and changing arch section from rectangle to shape composed of two trapezoids. Curves are given for determining the substitute section and its properties. The arch analysis based on these relations is briefly indicated and explained. (The same arch has been treated in *Beton u. Eisen*, 1922, p. 252, *Der Bauingenieur*, 1922, pp. 239 and 497, assuming a constant E.) One of the figures shows, by means of "contour" lines drawn on the arch elevation, variation of unit stress and variation of E. To obtain, by means of analysis based on straight line theory of stress strain variation, same results as are obtained by means of analysis based on parabolic variation, usual E value is multiplied by a correction factor which varies with type of loading, shape of arch, and range of temperature change. In two cases considered this factor was 0.64 and 0.85, indicating the unsuitability of using the usual high values of E obtained from compressive tests. Stress-strain curve tends to straighten out with increase in age of concrete. Tests by Mehmel show that same thing occurs under repeated loads. A reduction of 20 per cent to 30 per cent from original E value for the lower stresses may result from repeated loads. —A. A. BRIELMAIER

Bridge construction. "Der Brueckenbau." (See FIELD CONSTRUCTION.)

BUILDINGS

Concrete unit meets need for light weight roofing. *Concrete*, Feb. 1930, V. 36, No. 2, p. 40.—New York Coliseum, with central section 150 ft. wide and 450 ft. long has curved roof supported by trusses 20 ft. apart, resting on steel columns. Extreme height is 70 ft. Necessary light weight fireproof covering was provided by T-shaped concrete units known as Tee Stone. The units were 16 in. wide, 3½ in. deep with 1-in. flange and 1¼-in. stem. Lengths were about 8 ft. and weight per sq. ft. 15 lb. These units fulfilled requirements as to weight, strength, fireproof qualities and low cost.—F. E. RICHART

Reinforced concrete design simplified. *Concrete*, Feb. 1930, V. 36, No. 2, pp. 43-46. Equations and charts are given for the design of beams reinforced for compression. The A. C. I. Joint Building Code is followed.—F. E. RICHART

Reinforced concrete columns with cast iron cores. *Eng. News-Record*, Feb. 13, 1930, V. 104, No. 7, p. 277-278.—The core of Emperger type of reinforced concrete column consists of a solid cast-iron bar. Spiral reinforcing attached to vertical bars is provided near outside edge of column. Spiral represents about 1 to 2 per cent of cross sectional area of the column, vertical bars about 1 to 4 per cent, and cast iron core about 5 to 30 per cent depending on design. Chief advantage of this type of column over the usual reinforced-concrete column is the reduction in column size. By varying size of core, the outside dimension of column can be kept same for a number of stories thus resulting in a considerable saving in formwork. Tests by Austrian Concrete Committee are reported to have shown that cast-iron cores embedded in hooped concrete carried twice as much as structural steel in similar concrete. Cast-iron cores are cheaper and can usually be obtained in less time than structural steel of heavy section. Castings are made with rough surface to effect good bond between concrete and steel. In design, the columns are treated as reinforced-concrete columns to which concrete beams are connected in the usual way, no attempt being made to connect horizontal reinforcing to cast iron core of the column. The article includes information relative to building code provisions and test results for such columns.—D. E. LARSON

Calculation of flat slabs in terms of the moment in one direction. KURT BERNHARD. *Zement*, Dec. 1929, V. 18, No. 47, 49, 50, p. 1375-80, 1417-21, 1442-46. Review, *Beton u. Eisen*, Jan. 5, 1930, Heft 1, V. 29.—Methods and tables developed by Marcus and others for flat slab analysis and design have simplified computations to a certain extent, but still require considerable work on part of designer. This article carries simplifying of computations one step further. Two tables are given, from which design factors may be found from value of moment in one direction. A general explanation of the method of calculation and its theoretical basis is made. Practical application of method is illustrated by numerical examples. The tables may be of use in checking flat slab designs.—A. A. BRIELMAIER

Cross girder construction in the practice of reinforced concrete construction. STEPHAN SZEGOE. *Zement* (Germany), Jan. 1930, V. 19, No. 2, p. 34-7.—The construction with all kinds of cross slabs is described.—A. E. BEITLICH

Fireproof casing for iron. WENDT. *Zement* (Germany), Feb. 1930, V. 19, No. 6, p. 126-7.—Several materials and methods are described for fireproofing iron columns. An air space between column and casing is not advantageous. Air passage through interior of hollow columns is better. Best casing is strongly reinforced concrete without any air space. Masonry is destroyed very easily.—A. E. BEITLICH

Recent terminal improvements of the Boston & Maine Railroad. F. C. SHEPHERD. *J. Boston Soc. C. E.*, Jan. 1930, p. 1-28.—Development of terminal facilities of Boston & Maine Railroad in vicinity of Boston is described in detail, and plans for near future outlined. New construction started in 1926, and includes among larger projects two large classification yards in which switching is done by gravity, and classification is controlled by electro-pneumatic apparatus; new smokeless type engine terminal; power plant; automatic sand drying plant; and new North Station. Latter development includes heated concourse; mailing room; mezzanine floor for stores; coliseum over station, identical in size to the Madison Square Garden in New York; and three-story section for baggage and office space. Floor of coliseum is provided with complete brine piping system made of 1¼-in. piping embedded in special concrete to provide ice floor surface when desired. Construction of North

Station is fireproof throughout, and required about 1500 concrete piles, 15,000 cu. yd. concrete, 800 tons reinforcing steel, and 4000 tons structural steel.—MILES N. CLAIR

DAMS

Power station on the river Danube and on the Lake Neusiedel (Austria). FRITZ HOFFMANN AND GIOVANNI DEPERIS. *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Jan. 1930, V. 82, No. 1-2, p. 4-5.—Description of a big power plant with three turbines. Output of each turbine is 10000 K. W. Details are given of the construction of the dam and the locks.—A. E. BEITLICH

The big power plant on the river Dnjepr. WERNER KRUEGER. *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Jan. 1930, V. 82, No. 1-2, p. 5-6.—Design and description of big power plant and dam. Length of crown is 766 m. and its height 38 m. A lake is formed with surface of 270 km.^{2} On left hand side is passage for ships consisting of three locks each 120 m. long and 17 m. wide. Difference between levels is 37 m. Construction of dam consumed 730000 cu. meters concrete, and including locks and the power house over one million cu. meters. Power plant on right hand side is designed for an output of 350000 H. P. and with a possibility for enlarging up to 650000 H. P.—A. E. BEITLICH

MISCELLANEOUS

Buried reservoir design. DAVID LANDAU. *Concrete*, Feb. 1930, V. 36, No. 2, p. 37-39.—In article on the design of underground water reservoirs of reinforced concrete, continued from the October 1929 issue, equations are given for design of dome shaped bottoms and roofs as well as of flat bottoms. Methods of calculating general properties of reservoirs and of designing side walls are given, together with illustrative examples.—F. E. RICHART

Equalizing tank with 660 cubic meters capacity of the water works of Oldenburg i. o. (Germany). F. HIRT. *Zement* (Germany), Jan. 1930, V. 19, No. 5, p. 111-2.—Before entering water tower water passes an equalizing tank, constructed of reinforced concrete. Tank is divided by concrete wall into two compartments. Only one is in use at a time while the other can be cleaned. Water is fed to tank by means of an overflow. Floor is flat slab construction. Special features of structure are described and dimensions of tank are given.—A. E. BEITLICH

Concrete ceilings with refractory linings. HARRAEUS. *Zement* (Germany), Feb. 1930, V. 19, No. 6, p. 130-1.—Design and construction of concrete ceilings for furnaces with refractory linings are described and advantages of different forms of bricks discussed.—A. E. BEITLICH

Concrete coal bin for the boiler house of the new heat plant at the railroad station in Breslau. *Zement* (Germany), Feb. 1930, V. 19, No. 6, p. 131-2.—General description.—A. E. BEITLICH

Calculations of torsion and shearing stresses in reinforced concrete. "Berechnung des Eisenbetons gegen Verdrehung (Torsion) und Abscheren." ERNST RAUSCH. 1929, Julius Springer. R. M. 5.00 Reviewed in *Zement* (Germany), Feb. 1930, V. 19, No. 6, p. 134.—Simple formulas are given for the calculation of shearing stresses under torsion in shafts of various cross-sections in reinforced concrete construction.—A. E. BEITLICH

Pere Marquette Railway extends its concrete roadbed. PAUL CHIPMAN. *Eng. News-Record*, Feb. 27, 1930 V. 104, No. 9, p. 365-366.—Present 390-ft. extension of concrete roadbed built in 1926, and which has proven very satisfactory, (cf. *Eng. News-Record*, Feb. 24, 1927, p. 317, and Jan. 26, 1928, p. 151) embodies new design features which reduce cost by 35 per cent without sacrificing strength, allow lateral adjustment of rails, and eliminate rigidity and noises. Method of restoring surface to the bed has been

provided. Both 1926 and 1929 type of concrete roadbed are superior to the usual type of roadbed in that they have greater safety, higher permissible speed, lower track maintenance cost, and probable lower cost of maintaining equipment.—N. H. ROY

Strength of steel beams encased in concrete. P. W. LEISNER. *Concrete*, Feb. 1930, V. 36, No. 2, p. 29-30. —In design of steel beams encased in concrete, usual practice in this country has been to consider concrete only as fireproofing or protection against weathering, and to use structural steel members strong enough to support entire load without aid from the concrete. In Europe, common action of concrete and structural steel has long been recognized and has constituted a part of general practice, even in the design of railway bridges and culverts. German and Swiss railroads have been pioneers in use of concrete-encased I-beams. Their engineering departments investigated characteristics of these structures as early as 1901 by measuring deflections under live load. In 1903, Danish State Railways built a culvert with clear span of 10 ft. 6 in. construction consisting of old but usable I-beams encased in concrete. I-beams were German steel, about 10 in. deep, spaced about 20 in. on centers. Top of beams was about 3 in. below top of the concrete; bottom of beams was flush with concrete. First deflections under live load were measured 100 days after concrete had been cast. Deflection was about one-tenth of computed value for the unencased beams. Practically same results were obtained in tests made five months later, after culvert had been crossed by ten trains daily for five months. In *Teknisk Tidskrift* (Sweden), (Vag-och Vattenbyggnadskonst), 1923, No. 30, p. 22, reference is found to concrete-encased I-beam culvert of 12 ft. span built in 1905. After 18 years of service with about 40 trains crossing daily, accurate measurements of deflection under live load showed the value was about one-ninth of computed deflection for unencased I-beams. In the design department of the Swiss Federal Railways, strength of the concrete is accounted for by using high unit working stress and figuring the steel action alone.—D. E. LARSON

Baldwin filtration plant, Cleveland, Ohio. J. W. ELLMS, G. W. HAMLIN, A. G. LEVY AND J. E. A. LINDERS. *Proc. Am. Soc. C. E.*, Feb. 1930, V. 56, No. 2, p. 201-260.—The paper describes ten million dollar filtration plant built to serve Cleveland and additional Metropolitan District of about 500 sq. mi., considering history, hydraulics, design, construction, equipment and costs. Project includes largest covered concrete reservoir in the world as one of several concrete structures involved. Reinforced concrete design was based upon stresses of 16,000 and 650 lb. per sq. in. in steel and concrete respectively. Expansion joints extend down through foundation at points of pronounced change in the general type of structure for chemical house and mixing flumes, also at other points to insure maximum spacing of 82 ft. between joints. Other structures had expansion joints spaced from 60 to 82 ft. Four coagulation basins have 1041 20-in. square columns spaced 15 ft. 9 in. on centers which support groined arch roof. From floor of basins to crown of arch at centers of the basins is 22 ft. Column footings are independent of floor system which consists of 4-in. subfloor and 8-in. finished floor, latter reinforced with ½-in. square deformed bars on 15-in. centers each way. Columns are of five different lengths from 10 ft. 8 in. to 17 ft. 11 in, the two shorter lengths being unreinforced. Stability of floor designs was investigated on the basis of upward hydrostatic pressure equal to one third height of the water in the basin. Baldwin Reservoir is about 1035 by 550 ft. in plan and 40 ft. from floor to crown of groined arch roof, with total capacity of 135,000,000 gal. There are 1196 30 in. round tied columns 39 ft. 3 in. in height. The subfloor was 3 in. thick without expansion joints and the finished floor was 9 in. thick with prepared asphalt filler joints on center lines of columns (20 ft. 3½ in. apart). No steel was used in the finished floor except over drains. Two inch protection was used for column steel. The total load at base of each column was

211,000 lb. Groined arch roof having average thickness of $8\frac{3}{16}$ in. was selected for economy. Construction plant for Baldwin Reservoir included electrically driven 15-ton cableway, with two 85-ft. wood terminal towers operated on standard freight car wheels and axles. Large well arranged central mixing plant included two 1-yd. mixers discharging into hopper of 9 cu. yd. capacity. The 7 yd. buckets were filled from bottom slide gate and transported on narrow gage cars between cableway and mixing plant. Except for columns, all forms were made of $\frac{7}{8}$ in. center-matched lumber nailed to 2 by 6 in. studs, 16 in. on centers. Vertical forms were braced by 4 by 6 in. belt courses outside of studding and $\frac{5}{8}$ in. bolts running through section to be concreted. Column forms were steel, assembled in halves. Cableway handled all forms. All roof formwork was supported on bands about the columns which were previously poured up to the base of the groined arches. All formwork was made up in panels to permit repeated use.—H. J. GILKEY

Beauharnois hydro power development. *Can. Eng.*, Jan. 7, 1930, V. 58, No. 1, p. 101.—Beauharnois Hydro Power development is being made on the St. Lawrence River in Quebec, about 28 miles from Montreal at the lower end of a succession of falls between the sections known as Lake St. Francis and Lake St. Louis. Utilizing at some future time the entire flow with a drop of 80 feet, there will be developed approximately two million horse power. Water will be conveyed through a canal, which will also serve as a ship canal 27 ft. deep and 200 yds. wide with the power house at the Lake St. Francis end. Present construction will provide for the development of 500,000 horse power. When necessary concrete locks are installed, all of the difficult all-Canadian sections of the St. Lawrence waterway will be completed except the Lachine Rapids section. Construction requirements at Beauharnois are relatively simple, the canal excavation consisting of clay with the only rock formation at Lake St. Louis end where the power house will be situated.—G. M. WILLIAMS

Notes on the foundations of a new Thames-side warehouse. *Str. Eng.* (England), Jan. 1930, p. 17-19.—Discussion of Mr. Burns' paper, read before the Inst. Structural Engineers on Nov. 28, 1929, revealed that while specifications had required a 1:2:4 mix with a maximum slump of $3\frac{1}{2}$ in., the contractor had at his own expense changed the proportions to permit a 6-in. slump while maintaining the same strength requirement. Considerable discussion related to the exact nature and capacity of the soil upon which the foundations had been built.—V. P. JENSEN

ARCHITECTURAL DESIGN

Reinforced concrete and glass. *Zement* (Germany), Feb. 1930, V. 19, No. 6, p. 132-3.—Combination of glass and concrete in construction of big buildings yields beautiful architectonical effects. Use of glass is especially suited for advertising purposes.—A. E. BEITLICH

Norton memorial hall. M. A. ROLFE. *The Western Architect*, Feb. 1930, V. 39, No. 2, p. 26-27.—Monolithic concrete with decorative sculpture *poured* integrally with the building is unusual answer to the architect's problem of meeting client's needs without overstepping appropriation. This has been done on the Norton Memorial Hall, Chautauqua, New York, of which Otis F. Johnson of Chicago is architect. The donor stipulated that architectural design should be monumental in character, and that permanent buliding materials should be used. The Institute asked for an auditorium seating approximately fifteen hundred people, with fully equipped stage, dressing rooms, rehearsal rooms, property rooms, etc.

To the architect there seemed to be but one material available—monolithic concrete. Lorado Taft, aesthetic advisor, was familiar with possibilities of this material as medium for monumental sculpture, since he had used it for his "Blackhawk" monument at

Oregon, Illinois, and for his "Fountain of Time" in Chicago. The architect was familiar with its possibilities as a suitable building material through study of the work of Allison and Allison and of others upon Pacific Coast. To the layman, however, this material is often considered suitable only for foundations, bridges, and heavy engineering work. It was, therefore, somewhat of a surprise that the donors had no great objection to its use. Building was designed as four-square unit, treatment being consistent throughout, with sculptural ornament only upon the facade. It is approximately 83 ft. by 143 ft. with 40 ft. walls. Stage section is 28 ft. deep with a proscenium opening 38 ft. wide and 20 ft. high. Main floor of auditorium seats one thousand people, the balcony five hundred. Dressing rooms are provided on stage level and in two stories above. Concrete was placed in wooden forms. Work was divided into three operations —stage section, lobby section, and connecting auditorium walls. Reinforced concrete for balcony was not placed until building was enclosed. This was carried on piers and cantilevered over to side walls without being attached to them, to allow for expansion of materials. Considerable care was taken in form work, since it was desired to leave walls, both inside and out, as they came from forms. White pine dressed and matched lumber was used for this purpose, boards being carried horizontally around building and mitre joints being made at all exterior angles. Models for sculpture were made full size and molds made in plaster, strongly reinforced with hemp fibre. Two-by-fours were placed vertically on the backs of them for safety in transportation and ease in securing them in form-work. Sculpture was in every case poured as integral part of walls and in same operation. Experimental castings from molds were made before actual pouring. One difficulty thus discovered was need of preparing surface of the mold to prevent concrete adhering to plaster and causing extra labor in cleaning finished casting. Another was natural settlement of concrete in process of setting. This caused material to pull away from under side of the relief, thus distorting it. This had to be overcome by pouring very slowly in operations not to exceed eighteen inches an hour. Another difficulty was segregation of aggregates as they dropped, during the placing, through the reinforcing steel, causing the heavier material to fall to front, and resulting in stone pockets. This was overcome by conducting the concrete to within a foot or two of level of the *pour* by means of metal chutes. No special facing of finer material was used for sculpture, which as well as the exterior wall, received no further treatment after form and molds were removed, except usual cleaning with brush and water. The interior of building was also left as it came from the forms, except for two coats of cement paint to match color of plastered walls and ceiling. Acoustical difficulties were corrected by application of acoustical plaster to ceiling of auditorium.—REXFORD NEWCOMB

FIELD CONSTRUCTION

BRIDGES

Bridge construction. "Der Brueckenbau." A. SCHAU. 4th Ed., B. G. Teubner (Leipzig). R. M. 6.60. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Jan. 1930, V. 82, No. 3-4, p. 29.—Book of 216 pages discusses all kinds of bridge construction with different materials, especially concrete bridges. Examples and cost calculations are given.—A. E. BEITLICH

Overhead trusses carry arch centering for Pennsylvania R. R. bridge at Philadelphia. (See ENGINEERING DESIGN.)

BUILDINGS

How to save in concrete form work. *Concrete*, Feb. 1930, V. 36, No. 2, p. 19.—Pressure on column forms is resisted by variously detailed yokes clamped on form boards and holding them in tight grip. Yokes are spaced much closer at bottom of column than

at top. Two uppermost yokes are spaced 24 in. apart. Next one below is spaced at 20 in.; three at 18 in. each; three at 15 in.; three at 12 in.; and three more at 10 in.; provided length of column requires. To determine spacing for a column of any given height consider length of column as being measured from top. When dealing with octagonal columns, yokes are spaced similarly, using $\frac{3}{8}$-in. by $2\frac{1}{2}$-in. iron straps instead of 2 by 4 in. or 3 by 4 in. wooden yokes.—C. BACHMANN

The structures on the bathing beach of Karlsruhe on the river Rhine near Rappenwoerth (Germany). AMANN AND PLAESTERER. *Zement* (Germany), Jan. 1930, V. 19, No. 5, p. 103-7.—Description of the construction of a number of buildings and a bridge in reinforced concrete on a modern bathing beach.—A. E. BEITLICH

Winter jobs in reinforced concrete construction. LUZ DAVID. *Zement* (Germany), Jan. 1930, V. 19, No. 4, p. 79-85.—Possibilities for construction of buildings of reinforced concrete under cold weather conditions are demonstrated. Cost calculations for several cases in practice are included. Heating and thawing of raw materials in housed bins are described and a formula is derived for calculation of temperature of mixtures. Instructions are given for practical lubrication of the equipment and for the protection of concrete over night by covering and heating.—A. E. BEITLICH

The construction of superstructures. "Die Konstruktion von Hochbauten." OTTO FRICK AND KARL KNOELL. 1929, 10-11 Ed. B. G. Teubner (Leipzig). R. M. 9.60. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Jan. 1930, V. 82, No. 5-6, p. 47.—The book with its 368 pages discusses the various kinds of building construction; especial consideration is given to heat economy.—A. E. BEITLICH

Norton Memorial Hall. (See ARCHITECTURAL DESIGN.)

DAMS

The development of concrete construction in the United States of America. H. GRIESEL. *Zement* (Germany), Jan. 1930, V. 19, No. 5, p. 107-11.—A description is given of the developments in the concrete technic in the construction of the dams, Diablo (Wash.), Calderwood (Tenn.), and Kenogami (Que.). (cf. *Eng. News-Record*, Aug. 29, 1929, p. 320 and Oct. 24, 1929, p. 641.)—A. E. BEITLICH

MISCELLANEOUS

A new swing crane for construction jobs. FR. NELLISSEN. *Zement* (Germany), Jan. 1930, V. 19, No. 3, p. 61-2.—A movable crane, operated by compressed air, is described. The crane can lift and carry stones up to 150 kg. and is very useful on construction job.—A. E. BEITLICH

Specifications for mortars and concrete. (See MATERIALS.)

Baldwin filtration plant, Cleveland, Ohio. (See ENGINEERING DESIGN.)

The lock in the river Neckar at Heidelberg, Germany. *Zement* (Germany), Jan. 1930, V. 19, No. 1, p. 8-9.—A short description of the construction of a lock in reinforced concrete.—A. E. BEITLICH

Construction with reinforced concrete. "Der Eisenbetonbau." (Germany). C. KERSTEN. Part 2, 12th edition, Wilhelm Ernst and Son, 1929. Reviewed in *Zement* (Germany), Dec. 1929, V. 18, No. 50, p. 1449.—The book shows every possibility for the application of reinforced concrete to construction. Five sections deal with the following subjects: Construction of (1) buildings, the different kinds of ceilings, outer-walls, partition-walls, roofs, halls, joists; (2) foundations; (3) concrete pipes, containers for liquids and storage bins; (4) dams, dikes and caissons; (5) roads and bridges and industrial buildings.—A. E. BEITLICH

A device for the control of the efficiency of construction engines. SCHNEIDER-ARNOLDI. *Zement* (Ger-

many), Jan. 1930, V. 19, No. 2, p. 32-4. —A device in form of a clock can be attached to any movable part of an engine and every movement is automatically registerd. Its chief application is for elevators.—A. E. BEITLICH

Promotion and sale of ready-mixed concrete. ARTHUR C. AVRIL. *Rock Products*, Jan. 4, 1930, V. 33, No. 1, p. 157-161.—Plant description of the Avril Tru-Batch Concrete, Inc., Cincinnati, Ohio, and the methods used to promote use of ready-mixed concrete. Six factors contribute to successful operation: (1) Deliveries must be made at the time specified and on a precise schedule. (2) Concrete must be discharged from the trucks in good workable condition and it must be of a proper, uniform consistency to insure a perfectly smooth, dense structure if carefully placed. (3) Batching and mixing of the concrete must be done under accurate plant control. (4) Different mixes must be designed in accordance with intelligent, scientific procedure, combined with practical experience. (5) Plant should be designed to need very few men in its operation. (6) Efficient heating plant is absolutely necessary for winter business. Methods of maintaining concrete control, handling frozen aggregate, delivery fleet production records and forms are given. —J. J. LANDY

Self-unloading bulk cement cars. *Rock Products*, Jan. 4, 1930, V. 33, No. 1, p. 150. All-steel bulk cement rail carriers with special screw conveyor unloading devices designed for shipment to metropolitan areas or to contract jobs. Three illustrations.—J. J. LANDY

Construction engines. "Baumaschinen." HANS FEIHL. (Germany), 1929. R. Oldenbourg, Muenchen and Berlin. Reviewed in *Zement* (Germany), Jan. 1930, V. 19, No. 3, p. 64.—In six sections are described all kinds of machinery which are used in construction work. Tables are given to facilitate their design and construction. —A. E. BEITLICH

Lining and scaffolding. "Schalung und Ruestung." FRANZ BOEHM. (Germany.) Wilhelm Ernst und Son, 1929. Reviewed in *Zement* (Germany), Dec. 1929, V. 18, p. 1402. —Book of 132 pages gives instructions for form linings and their transportation, and scaffolding, showing how unnecessary costs can be prevented.—A. E. BEITLICH

Cost calculation for engineering construction. "Kostenberechnung im Ingenieurbau." (Germany.) HUGO RITTER. 2nd Edition, 1929. Julius Springer. Reviewed in *Zement* (Germany), Dec. 1929, V. 18, No. 52, p. 1475.—The book helps the contractor to calculate in advance the costs for any construction program including transportation of materials, engines, digging, concrete, masonry, woodwork, pipe laying, pavements, tools and other costs.—A. E. BEITLICH

Advances in structural concrete in 1929. *Concrete*, Feb. 1930, V. 36, No. 2, p. 13.—Use of ready-mixed concrete on structural work made most decided advance in 1929. A Philadelphia concern sold 250,000 cu. yd. in that city, and is installing additional equipment to take care of an expected volume of 400,000 cu. yd. in 1930. It is now necessary to make distinction between concrete centrally mixed and mixed in transit. Former is mixed at central plant, discharged into trucks and hauled to destination in mixed form. Latter is mixed in specially designed concrete mixers mounted on trucks, actual mixing taking place while truck is in transit from material storage bins to place of delivery. Tests of far-reaching importance, to determine increased carrying capacity of steel beams when encased in concrete, were conducted during latter half of 1929 at Gary plant of the American Bridge Co. Observers learned that the concrete added greatly to the strength of beams. Prospects for structural concrete work in 1930 may be considered in light forecasts relating to general construction prospects. General opinion seems to support prediction structural work in general, and concrete work in particular, will at least equal corresponding figures of 1929.—C. BACHMANN

Railway uses central mixing plant on 17-mile project. F. L. RIDDLE. *Concrete*, Feb. 1930, V. 36, No. 2, p. 27.—Connellsville Extension of Pittsburgh & West Virginia Railway is probably first new-line railway construction project on which centrally mixed concrete has been used exclusively. It will require about 60,000 cu. yds. of concrete in construction of bridges and tunnel lining, major structures consisting of one crossing of Monongahela river and two crossings of Youghiogheny river. During summer of 1929 a stretch of 17½ miles of line was under construction. Three classes of concrete were placed according to water-cement ratio specification. Classification was controlled by cylinders broken at 7 and 28 days, minimum requirements being 3,000 lb. per sq. in. for Class A concrete, 2,500 lb. for Class B, and 2,000 lb. for Class C. One of greatest advantages of central mixing plant was ease with which aggregates could be assembled and stored. Another major advantage arises from fact that large permanent mixing plant can be established where most approved equipment and methods for control of the concrete can be installed. Third important and obvious advantage comes from fact that central mixing plant enables railway company to inspect concrete at one source, thereby eliminating the need for placing inspectors at each structure. Concrete was delivered to various structures in high-speed trucks. Greatest time required for delivery from mixer to point of use was about 45 minutes.—C. BACHMANN

ROADS AND PAVEMENTS

Handbook of new road construction methods with bitumen, tar and portland cement. "Handbuch der neuen Strassenbauweisen mit Bitumen, Teer und Portland Zement als Bindemittel." W. REINER. 1929, 1st Ed. Julius Springer. R. M. 30.50. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Jan. 1930, V. 82, No. 3-4, p. 29.—Book is most complete work dealing with modern road construction. Different test methods and effect of static and dynamic forces on surface of pavements are characterized.—A. E. BEITLICH

Highway research in 1929. R. W. CRUM. *Rock Products*, Jan. 4, 1930, V. 33, No. 1, p. 153-154.—Summaries of principal developments and experimental work.—J. J. LANDY

Accuracy of water measurement on paving mixers.* BERT MYERS AND F. C. LANG—Two series of tests were made to determine the accuracy of water measuring devices used on paving mixers, the first being a field study made by the Iowa Highway Commission and the second a laboratory study made by the Minnesota Highway Department. In both of these series the effect of the following variables on the accuracy of the measurement were studied: (1) effect of air entrapped in measuring system, (2) overlapping in the opening cycles of the control valves, (3) variable pipe line pressures and (4) personal equation of the operator. In addition, the laboratory study included consideration of the effect of variation of grade and super-elevation as well as time of filling and discharging on accuracy of measurement. Results of both of these investigations emphasize the need for improvement and refinement of design and construction of equipment used for measurement of water for concrete mixers. One prominent defect of the measuring devices observed was in the three-way valve, unless this valve was automatically controlled. The mixer operator could inject into mixer drum any additional amount of water, over and above that set on the indicator, by holding the valve lever in an intermediate position between the "fill" and "discharge" positions. This was possible because both intake and discharge valves were open at same time and an unmeasured quantity of water was passed from supply line directly to mixer drum. With rapid operation of the valves this measured quantity is small, but increases with slowing of the valve operation and increase in pipe line pressure. This same criticism applies to connected gate valves not so interlocked as to prevent one valve opening before other is completely closed. Quantity

*Presented before the 9th annual meeting of the Highway Research Board, Washington, D. C., December 13, 1929. Abstracted in advance of publication.

of water delivered to drum was also affected by superelevation and grade. This effect was much greater in tanks mounted with the axis horizontal than for tanks mounted with the axis vertical, because it is practically impossible to design an adjustable draw-off or displacement device which will provide for delivery of a constant quantity of water from a tank whose axis is horizontal for all normal positions of the tank and all settings of the control device. Effect of variations in pipe line pressure may be eliminated by interposing a supplementary tank open to the atmosphere from which water will be fed to measuring tank by gravity. Air entrapped in measuring tank may be removed by substituting for air relief valves a riser pipe with a height equal to that of top of the supplementary tank. One other improvement in connection with operation of valves that may be worthy of consideration would be to so arrange the mechanism that it would be difficult for operator to introduce into the mixer drum a quantity of water in addition to measured quantity or make it impossible to do this without the knowledge of the inspector. Attention is called to the fact that mixer manufacturers have taken note of results of these studies and have made many of the improvements suggested. Valves have been improved to prevent passing of water directly from pipe line into the mixer drum. Vertical tanks have been substituted for horizontal tanks. Indicating devices have been improved and several manufacturers are prepared to furnish mixers equipped with supplementary tanks so arranged that water is fed to the measuring tank by gravity and a riser pipe open to the air is substituted for air relief valves.

Improvements which will eliminate principal sources of error in water measurement can be made at a nominal expense on mixers now in use. The enforcement of specifications requiring accuracy of water measurement will no doubt stimulate further improvements in design of water measuring devices. —F. H. JACKSON

Sheet concrete pavement placed on Ambassador bridge approaches. *Concrete*, Jan. 1930, V. 36, No. 1, p. 27. —Pavement for the approaches to the first span of land section of the Ambassador bridge across the Detroit River, connecting Ontario with Detroit, Mich., was constructed of concrete by the sheet concrete method to provide a resurfaceable concrete pavement on earth fill. Method followed consisted of laying a 7-in. base course of concrete and a 2-in. top separated from the base by a sheet of coarsely woven burlap, making the total thickness of the slab 9 in. Pavement was built in rectangular sections, side forms being set to the proper elevation and grade. These sections were 30 ft. long and 11 ft. 9 in. wide, containing approximately forty square yards of 9-in. concrete pavement, or ten cubic yards of concrete each. As soon as the top course was dumped from the concrete buggies and spread over the steel mat, the mat was removed by working it to the top of the concrete and then lifting it out of the way bodily. The side forms for the panels were constructed to provide a keyed joint between adjoining panels. —C. BACHMANN

SHOP MANUFACTURE

Review shows concrete products invading new fields. *Concrete*, Feb. 1930, V. 36, No. 2, p. 16.—Recent survey of more than 4,000 concrete products plants, representing about 90 per cent of total volume, indicated that decrease in concrete masonry in 1929 as compared with 1928 was less than half of decrease in the residential building field. This indicates concrete masonry was used in much larger percentage of non-residential construction during past year than ever before. The survey of concrete products plants also revealed largest decrease in operating plants in any year since record of industry has been kept. From 1919 to 1928, decrease in plants totaled 40 per cent. In 1929 approximately 15 per cent less plants were in operation than in 1928. The Cast Stone Institute has made more progress in 1929 than any other

branch of the concrete products industry. Strong competition in concrete burial vault field led group of Ohio burial vault manufacturers to start work on standard specification regulating quality, resulting in marked improvements in quality of concrete.—C. BACHMANN

Testing cement colors. *Concrete Building Concrete Products*, (England), Jan. 1930, V. 5, No. 1, p. 6-7.—Suitability of mineral colors for mixing with cement to produce colored concrete depends upon staining power, fineness, freedom from efflorescence and light resistance. Reliable tests applicable even by small operator are as follows: (1) Test for contraction in which one part of color is mixed dry with two parts cement and distilled water added to make quaky consistency. Mixture is spread on glass and allowed to harden in air for 24 hrs. It is then placed under water and observed for cracks. (2) Test for fading—Distilled water is added to dry color and thoroughly shaken, making a creamy fluid. A suitable color will mix readily and not float. Chloride of lime is mixed with part of the creamy sample and both parts observed on glass for difference in color. Useless colors result in paler or mottled appearance. (3) Test for staining power—Small quantities of different colors are mixed with 10 volumes of cement by passing repeatedly through fine sieve. Mixtures are spread on glass and water added to make soft paste. After samples are dried, results are compared. (4) Test for fineness—Color is mixed with water and allowed to settle. Water is drained off and color dried by artificial heat. Inspection of broken part under magnifying glass will reveal coarse particles near bottom. (5) Test for light resistance—Pastes of color and distilled water sealed between two glass plates and part of pat is covered by black paper. After exposure to sun or strong artificial light, contrast between exposed and covered portions determines light resistance. (6) Test for aniline—Separate samples mixed with water, turpentine and alcohol are thoroughly shaken. The presence of color in any of the liquids after allowed to settle indicates presence of aniline.—D. F. JENNINGS

The presence of coal in concrete cinder blocks. Mixtures of fine ashes and clinkers. *Chem. Ind.*, (Germany), 1930, V. 23, p. 86.—Failures in cinder blocks are attributed to the presence of underburned material.—(From Portland Cement Ass'n Literature Supplement.)

Glazed concrete as decorative or facing material. O. WEDEGARTNER. *Brit. Sci. Abs.* (England), 1929, V. 2, No. 12, p. 434.—Several examples of the use of cold glazed cement products for decorative and facing purposes are illustrated and briefly described.—(From Portland Cement Ass'n Literature Supplement.)